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High-Speed Rail IDEA Program

Investigation of a Hybrid-Composite Beam System

Final Report for High-Speed Rail IDEA Project 23

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This investigation by Teng & Associates, Inc. was performed as part of the High-Speed Rail IDEA program, which fosters innovative methods and technology in support of the Federal Railroad Administration's (FRA) next-generation high-speed rail technology development program.

The High-Speed Rail IDEA program is one of four IDEA programs managed by TRB. The other IDEA programs are listed below.

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- Elliot Company of Indianapolis

The most rewarding conclusion of this project is that a mechanism exists within the transportation industry that allows for the opportunity to conceive an innovative idea and bring the concept to fruition in a manner that perpetuates the advancement of knowledge in the engineering profession. That being said, I would like to thank the HSR-IDEA Program Committee for taking a chance on a concept that might otherwise have been forgotten. Last but not least I would like to thank HSR-IDEA Project Manager, Chuck Taylor, for his invaluable guidance, patience and encouragement.

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1.0 EXECUTIVE SUMMARY

This High-Speed Rail IDEA project investigated the concept of a hybrid-composite beam as a structural member for use in railroad bridges. The beam is comprised of three main sub-components: a shell, compression reinforcement, and tension reinforcement. In the preferred configuration, the shell is comprised of a fiber reinforced plastic (FRP) box beam. The compression reinforcement consists of portland cement concrete which is pumped into a profiled conduit within the beam shell. The tension reinforcement consists of carbon, glass or steel fibers anchored at the ends of the compression reinforcement. The orientation of these sub-components is shown in Figure ES1. This report addresses cost metrics, design limit states and fabrication issues related to the manufacturing of the hybrid-composite beam. Finally, the report details the steps that were involved in the design, fabrication and load testing of the first prototype hybrid-composite beam.

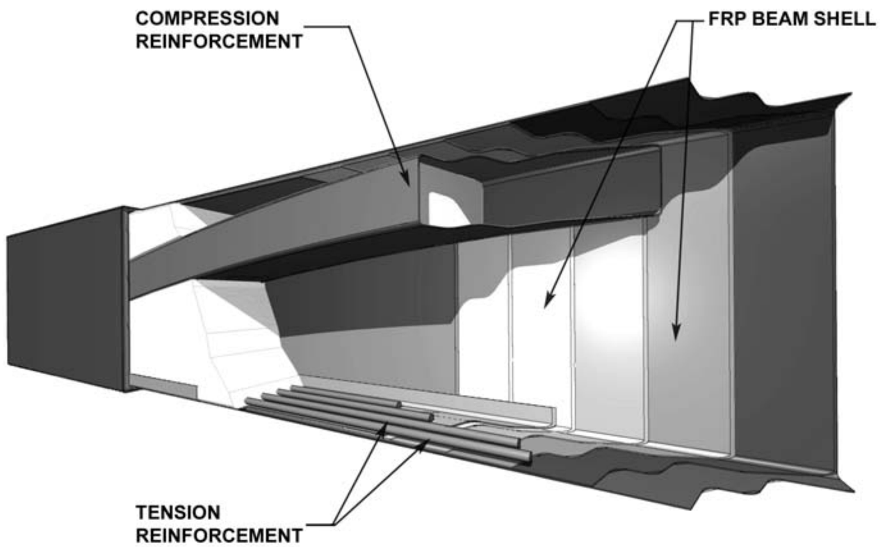


Figure ES1. Fragmentary Perspective of Hybrid-Composite Beam

The project findings documented that the hybrid-composite beam can be manufactured with minimum tooling costs and that the girders can be predictably designed to satisfy the strength and serviceability requirements of the American Railway Engineering and Maintenance of Way Association (AREMA). Cost metrics indicate that the hybrid-composite beam offers a cost-effective alternative to concrete or steel beams in a railroad bridge. What distinguishes the hybrid-composite girder from beams of conventional materials is that the FRP materials offer greater corrosion resistance and potentially longer life. Further, because of their reduced weight, shipping and erection costs for the hybrid-composite girders offer a distinct advantage. Finally, because satisfying serviceability requirements typically drives the design of the hybrid-composite girders, these girders can provide additional strength capacity beyond what is required by code. With these and other inherent benefits, the hybrid-composite girder offers an attractive alternative to consider in the construction of new railroad bridges as well as the reconstruction of the existing railroad bridge inventory.

The first step in the investigation was to establish the cost metrics associated with railroad bridge construction. Bridge designs of conventional materials were compiled to ascertain the costs of typical railroad bridges. In compiling these designs, a survey was conducted with the Class 1 railroads to ascertain the distribution of span lengths and material types within the existing railroad bridge inventories. This information was analyzed to determine the target market to focus on for the development of the hybrid-composite beam. As the other stages of the project were completed it became possible to develop comparable bridge designs using the hybrid-composite girders. These comparable designs were used to establish the cost metrics of the proposed IDEA product for comparison to conventional materials. Finally, life-cycle cost comparisons for railroad bridges of conventional materials versus hybrid-composite beams were conducted.

Composite manufacturing processes already exist that are well suited to the fabrication of the hybrid-composite beam. Despite this, the various components that define the preferred embodiment of the beam pose some interesting challenges in adapting these manufacturing processes to fabrication of a composite of this nature. During the early stages of the investigations a study was conducted to evaluate and select materials better suited to the preferred manufacturing process. The materials investigated include resins, glass fabrics, core materials, tension reinforcing, compression reinforcing materials and materials and methods to form the conduit inside of the beam for the compression reinforcing. Also during this phase, small experiments were performed with the tooling and a vacuum assisted resin transfer method (VARTM) to combine these various components into the end product. Although different combinations of materials could be used to manufacture the beam, a determination was made as to the materials and methods to be used for the prototype beam investigated in the experimental phase.

One of the keys to making FRP composites more readily acceptable in the transportation sector will be simplification and standardization of the design and analysis methods. Stage 2 of the investigation was devoted to deriving equations suitable for quantifying the structural behavior of the hybrid-composite beam. During this stage, the critical design limit states were established. A comprehensive spreadsheet was developed to facilitate quick parametric studies of the various design components to help develop and intuitive knowledge of the structural behavior. With this knowledge it was possible to prioritize the steps involved in design and ascertain the lay-up for the prototype test specimen.

The final stage in the investigation involved the manufacturing and testing of a prototype beam. In this stage, a 19-foot prototype beam was designed and fabricated to accommodate a ballasted deck and Cooper E-80 live loads. Once fabricated, this beam was placed in a loading frame and using hydraulic actuators was subjected to a battery of tests to simulate both service level and factored load conditions. The beam was subjected to both symmetrical and unsymmetrical loading patterns. The beam was also subjected to 100,000 cycles of fatigue loading before being loaded to failure at over 3.5 times the required design service load.

2.0 IDEA PRODUCT

2.1 PRODUCT DESCRIPTION

The product resulting from this investigation will be a composite beam system that provides lighter weight for transportation and erection with enhanced corrosion resistance. The proposed Plasticon-Optimized Composite Beam System, U.S. Patent No. 6,145,270, consists of three main sub-components. The first of these is the fiber reinforced plastic (FRP) beam shell, which encapsulates the other two sub-components. The second major sub-component is the compression reinforcement which consists of portland cement grout or concrete which is pumped or pressure injected into a continuous conduit fabricated into the beam shell. The third and final major sub-component of the beam is the tension reinforcement, which is used to equilibrate the internal forces in the compression reinforcing. This tension reinforcing could consist of unidirectional carbon or glass fibers or it could consist of steel fibers, e.g., standard mild reinforcing steel or prestressing strand infused in the same matrix during fabrication of the glass beam shell.

The beam shell is constructed of a vinyl ester resin reinforced by glass fibers optimally oriented to resist the anticipated forces in the beam. The beam shell includes a top flange, bottom flange and a continuous conduit. The conduit is fabricated into the shell and runs longitudinally and continuously between the ends of the beam along a profile that is designed to conform to the internal load path resisting the external forces applied to the beam. The beam shell also includes two vertical webs, which serve to transfer the applied loads to the composite beam as well as to transfer the shear forces between the compression reinforcement and tension reinforcement. All of the components of the beam shell are fabricated monolithically using a vacuum assisted resin transfer method (VARTM), but could be manufactured using other processes.

The compression reinforcement, which consists of portland cement concrete, but could consist of portland cement grout, polymer cement concrete or polymer concrete, is introduced into the conduit within the beam shell by pumping it through the injection port located in the side of the conduit. The profile of the compression reinforcement follows a parabolic profile which starts at the bottom corners of the beam ends and reaches an apex at the center of the beam such that the conduit is tangent to the top flange. The profile of the compression reinforcement is designed to resist the compression and shear forces resulting from vertical loads applied to the beam in much the same manner as an arch structure. While the preferred embodiment assumes introduction of the compression reinforcement after the beam shell has been erected, it could also be introduced during fabrication of the beam shell. The parabolic shape selected is ideally suited for simple span beams, however the profile of the compression reinforcement, and for that matter, the tension reinforcement can follow other profiles that are determined based on optimization to accommodate the internal load path. Variations in the profile might be more desirable on continuous span applications.

The thrust resisted by the compression reinforcement resulting from externally applied loads is equilibrated by the tension reinforcement. The tension reinforcement consists of layers of unidirectional carbon reinforcing fibers with a high tensile strength and high elastic modulus. In place of carbon fibers, glass fibers, aramid fibers, mild reinforcing steel or prestressing strand could also be used as tension reinforcement. The fibers, which are located just above the glass reinforcing of the bottom flange and along the insides of the bottoms of the webs, are oriented along the longitudinal axis of the composite beam. The tension reinforcement is fabricated monolithically into the composite beam at the same time the beam shell is constructed, but could also be installed by encasing conduits in the beam shell which would allow installation at a later date. The tension reinforcement could also be applied by bonding to the outside of the beam shell after fabrication. The method of external application can also be adapted for future strengthening or repairing damaged beams.

A bridge can be built quickly and easily using the beam described above. The composite beams are erected prior to injection of the compression reinforcement by placing them with a crane in the same manner as a steel or prestressed concrete beam. The composite beams are easily self-supporting prior to and during the installation of the compression reinforcement. In the case of bridge replacement or rehabilitation it may be possible to reuse existing abutments and/or intermediate piers. The compression reinforcement is then introduced into the composite beam by injecting portland cement concrete into the profiled conduit in the glass beam shell. The compression reinforcement is injected using pumping techniques, which are common to the construction industry. No temporary falsework is required for the erection of the composite beams or during the injection of the compression reinforcing.

Once the composite beams are in place and the compression reinforcement has been introduced, the deck pans, ballast and track work can be installed. The weight of the composite beams during transportation and erection is approximately one fifth of the weight of the conventional steel beam required for the same span and approximately one tenth of the weight required for a precast prestressed concrete beam for the same span.

2.2 POTENTIAL IMPACT

Based on the current rate of spending for bridge replacement and rehabilitation, the number of deficient bridges is only gradually being reduced. Given that the structural framing members comprise a significant portion of the overall cost of a bridge, this provides all the more reason to develop a product that can increase the service life of the infrastructure. The hybrid-composite beam investigated provides a new alternative for consideration in replacement of our deteriorating bridges. Some of the inherent benefits of the beam are as follows:

- Faster erection, resulting in less out of service time for highways and railways.
- Lightweight, allowing reuse of existing substructures and potentially reducing span replacement costs.
- Design governed by deflection criteria results in reserve capacity for strength. This can benefit short-line railroads as well as other bridges that require upgrading for additional capacity or heavier axle loads.
- Resilient materials can improve seismic performance and fatigue resistance.
- Corrosion resistant materials result in better life cycle costs.
- Development of advanced composite materials in other components of high-speed rail systems and highway bridges results in a synergy of materials that could ultimately lead to economies of scale.
- Bridge girder would not require cathodic protection or be prone to stray currents or magnetic fields created by electrical train systems or magnetically levitated train systems.

Because the beam presented uses basically the same erection methods as steel or precast concrete beams, contractors are subjected to virtually no learning curve despite the use of advanced composite materials. The beam can accommodate any alignment, skew or curve as a conventional beam but with the added feature of greater corrosion resistance. It is also well suited for bridges requiring future widening. Due to simple tooling and readily available raw materials, these beams are also well suited for emergency replacement of girders or entire spans on existing structures. Further, the embodiment of this beam lends itself well to implementation in current design codes such as AASHTO or AREMA, in limit states that can easily be followed by structural engineers with little or no background in FRP.

The ultimate objective of this investigation was to develop the hybrid-composite beam system to the level that it could be implemented in a commercial application. In order to be a viable alternate, the proposed beam must be competitive with prestressed concrete beams and steel beams both with respect to structural performance and cost performance. There already exists a tremendous market for structural beams in new bridge construction. However an equally if not larger market exists for the replacement and rehabilitation of bridge structures for both railroad bridges and highway bridge structures.

In order to quantify the potential market, a study was performed to compile select statistical data related to bridge inventories in the United States. For highway bridge structures, this task is relatively simple as the Federal Highway Administration maintains the U.S. Bridge Inventory, for record keeping purposes and to help evaluate and prioritize their capital program for bridge replacement and maintenance. For railroad bridge structures this task is a bit more complicated as no one source has a statistical database compiled for the inventory of railroad structures. However, most of this information does exist and an effort was made to contact all of the Class 1 Railroads, as well as some Short Line Railroads and the Association of American Railroads (AAR) to obtain this information. Shown below is the data reduction of the total lengths of railroad bridges as sorted by material. The quantities listed under the totals are based on the most recent compilation by the AAR.

Railroad	Steel	Concrete	Timber	Total
Norfolk Southern	1,272,480	309,144	168,854	**1,750,478**
Union Pacific	882,762	565,602	660,750	**2,109,114**
BNSF	882,000	458,000	519,000	**1,859,000**
Other Class 1 RR	2,462,758	467,254	1,451,396	**4,381,408**
Total	**5,500,000**	**1,800,000**	**2,800,000**	**10,100,000**

Table 1. Class 1 Railroad Bridge Inventories (Lengths in feet as categorized by Material)

Based on the information obtained in this investigation, there appears to be in excess of 10,100,000 linear feet of railroad bridges owned and maintained by the Class 1 Railroads in the United States. Of this total approximately 54% are constructed of steel, 18% concrete and 28% are constructed of timber. In general the older structures tend to be the ones constructed of steel and timber. When evaluating by span length, the majority of bridges less than 20 feet in length are timber bridges. Almost all of the bridges spanning over 50 feet are constructed of steel. The remainder of bridges spanning between 20 and 50 feet are relatively balanced between concrete and steel. As part of the survey, a breakdown of the bridge inventories by span length for each material type was also requested. A breakdown of the percentages by span length and material type is presented in Table 2.

Span Length	Steel Bridges	Concrete Bridges	Timber Bridges	Fraction of All Bridges	Extrapolated Length
0 to 20’	5.6	5.8	13.8	25.1	2,539,792
20’ to 30’	6.4	5.9	1.7	13.9	1,406,828
30’ to 40’	5.4	2.7	1.2	9.2	932,455
40’ to 50’	5.4	1.4	0.9	7.7	774,452
Over 50’	30.3	7.7	6.0	44.0	4,446,473
Total	**53.0**	**23.4**	**23.6**	**100.0**	**10,100,000**

Table 2. Span Length Distribution of Bridge Inventories

It should be noted that the data for the distribution of bridges by span length was limited to information obtained from only three of the Class 1 Railroads and subsequently represents approximately 57% of the total railroad bridge inventories for the United States. The fractions of all bridges were used to extrapolate the total length for each span range based on the ratio of the total bridge inventory to the sum of the bridge inventories submitted by the respondents.

Based on conversations with contractors and engineers involved in railroad bridge construction, it appears that many of the bridge replacements being done in recent years are comprised of short span timber bridges being replaced with prestressed concrete girder bridges. Almost all of the old timber bridges are less than 20 feet in length. In fact, many of these bridges are between 12 and 15 feet in length. Subsequently, by replacing the timber bridges with concrete, the objective is usually to double the span length on the new bridge versus the old bridge. Prestressed concrete beams tend to be more economical in the 30-foot span range as compared to steel. As a result, prestressed concrete box beams have evolved as one of the more popular structural members for railroad bridge replacement in recent years. Although the proposed hybrid-composite girder theoretically will work for a wide range of span lengths, the focus of this study was on the span range of 20 to 50 feet, with the target market being optimally a 30-foot span.

In terms of total market potential in the railroad industry, based on the inventory survey data, the Class 1 railroads are replacing on average 70,000 linear feet of bridge structure annually or less than 1% of the total inventory. Even at a replacement cost of approximately $3000/linear foot, which appears to be on the lower end of the spectrum, this results in a $210,000,000/year construction industry. In terms of the structural framing component by itself, an average cost might be more on the order of $1500/linear foot of track. In terms of specific applications of the proposed beam, the percentage of total potential market would need to be confined to some percentage of the total. Inevitably, there will be unique circumstances in some bridge projects that which may preclude the use of the proposed hybrid-composite girder system. Regardless, if the proposed IDEA product is feasible for applications up to at least 50 feet, this represents 56% of the total bridge inventory. Assuming the distribution of replacement structures is proportional to the span length distribution, structural framing replacement amounts to a $58,800,000 annual industry.

Although the market potential for railroad bridges is significant, the market potential for the hybrid-composite beam is further expanded by the highway bridge industry. According to the National Bridge Inventory as maintained by the Federal Highway Administration (FHWA), of the nearly 590,000 highway bridges in the United States, over 170,000 or 29% are either structurally deficient or functionally obsolete. It is more difficult with highway bridges to ascertain the specific potential for utilization of the proposed beam system in that there is more variation in bridge deck widths, span lengths and framing configurations than is typically found in railroad bridges. Regardless, according to the FHWA, the cost to improve the nation’s bridges to eliminate deficiencies is $10.6 billion a year for the next 20-year period [1]. This combined with the market for railroad bridge structures comprises a substantial potential market for this IDEA-Product.

3.0 CONCEPT AND INNOVATION

3.1 CURRENT TRENDS IN FRP INFRASTRUCTURE APPLICATIONS

Considerable work has been done in recent years with respect to FRP in the transportation infrastructure. In order to emphasize the unique characteristics of the hybrid-composite beam, it is important to point out that most of the research in advanced composite materials for transportation applications has been limited to structural shapes comprised of homogeneous FRP materials. Other research includes the application of FRP materials to conventional structural members to enhance strength. The following is a list of the various FRP research categories with respect to the bridge industry:

* Bridge framing systems using glass FRP pultruded beams or trusses manufactured from pultruded shapes.
* Glass FRP cable stayed pedestrian bridges fabricated from pultruded shapes with FRP cable stays.
* FRP reinforcing and post-tensioning strand for reinforcing and prestressing conventional concrete beams.
* Bonding FRP sheets to existing concrete and steel structures as a means of repairing, strengthening and upgrading these structures.
* FRP column wraps to provide confinement for enhanced seismic performance of concrete columns.
* Concrete filled, circular FRP tubes as an alternate to reinforced concrete columns.
* FRP bridge decks manufactured as pultruded sections, or VARTM sandwich panels.
* Hybrid pultruded beams using carbon reinforced flanges and glass reinforced webs.

In most cases, where bridges are constructed of structural members fabricated entirely of FRP, these bridges are subject to several constraints that have precluded widespread acceptance of the technology. First and foremost has been cost. The increased costs can be directly traced to raw material costs and constitutive material properties. In general, both glass and carbon have strength capacity that meets or greatly exceeds that of steel. However, they are much more flexible materials and require additional material to satisfy the serviceability requirements for deflections. The amount of material required and the higher material costs make it virtually impossible for a purely FRP beam to be cost competitive with concrete or steel beams at the present time.

Other limitations in application and span length in FRP beams are a result of lower shear strength capacity and low elastic buckling capacity in compression. Again combined with the flexible nature of these materials, applications of purely FRP bridges have generally been confined to pedestrian bridges and short span county bridges. In any case, these bridges are still more expensive than those using conventional materials. Although the increased service life can improve the life cycle costs, most clients are still partial to construction materials that offer lower first costs.

3.2 UNIQUENESS FOR APPLICATION

What distinguishes the hybrid-composite beam is that it uses conventional materials, i.e. concrete and steel, in conjunction with FRP components to create a structural member that exploits the inherent benefits of each material in such a manner to optimize the overall performance of the beam. Whereas FRP materials are generally too expensive and too flexible when arranged in a homogeneous form, the strength and stiffness are provided by a more efficient use of materials that are well suited to purely axial tension or compression. The classical arch shape of the compression reinforcing also helps reduce the shear carried by the FRP webs and ensures an optimal use of the compression and tension reinforcing.

Despite the unique configuration and diverse nature of the building materials used in the hybrid-composite beam, the end product does not result in any unique characteristics that would preclude the beam from being used in the same manner as more conventional framing systems. The cross-section is very conducive to standardization, similar to prestressed beams or rolled steel sections. Yet at the same time the tooling required for the VARTM manufacturing process is simplistic and inexpensive and allows for considerable variation with respect to overall dimensions, shape and internal lay-up. The embodiment of the hybrid-composite beam results in a beam that has several inherently unique benefits, while remaining cost competitive with conventional materials.

4.0 INVESTIGATION

4.1 COST METRICS

4.1.1 General Evaluation Criteria

In order to establish the viability of the hybrid-composite girder for the target market, it was necessary to quantify the cost metrics of the proposed beam relative to the cost metrics of competing conventional materials. The first step was to determine more specific parameters for the bridge designs. Bridges for all materials have been designed in accordance with the American Railway Engineering and Maintenance-of-Way Association (AREMA), Manual for Railway Engineering. The design live load for the bridges is the Cooper E-80 Load or the Alternative Live Load. All bridges in this study were designed with a 12-foot wide ballasted deck within the targeted span range of 20 to 50 feet.

The following is a brief description of the bridge designs investigated including both conventional materials and hybrid-composite girders. From these base designs it was possible to estimate the initial construction costs associated with each of the various competing materials. This data will be used in the Life-Cycle Cost analysis (LCC) to establish the relative costs of the competing materials over a predetermined service life for the structure. In general, the analysis and design of the representative bridges for each material type has been limited to design of the superstructure elements only. The configuration of the sub-structures, i.e. piers and foundations for any of the competing bridge materials would essentially be the same. The only variations arise from lighter dead loads for both steel and hybrid-composite girders relative to concrete girders that can result in potential cost savings for the substructure component.

4.1.2 Conventional Bridge Designs

The first conventional bridge material to be considered is structural steel. The development of the base designs in structural steel considered several parameters including span length, span to depth ratio, rolled-sections versus built-up plate girders and number of girders per track. The span to depth ratios considered were span/10 and span/15. In all cases, including rolled sections and plate girders, it was assumed that the girders were supporting a non-composite 8" concrete slab that serves to support 16" of ballast and rails. In total, 36 different steel bridge designs were developed to evaluate the relative costs associated with this type of framing system as well as develop a feel for how variations in the parameters considered can affect the efficiency of the design. Within the span ranges considered, the built-up plate girder sections with a span to depth ratio of span /10 appeared to be the more economical steel structures.

Another type of conventional material considered was reinforced concrete. In the bridge designs considered in this investigation, the cast-in-place concrete was reinforced with mild steel reinforcing bars. Many older concrete bridges were also designed and constructed utilizing concrete encased steel beams. This type of framing systems has not been considered in this investigation. It should be noted that although reinforced concrete still provides a relatively economical solution, most bridge replacements nowadays have to be conducted under traffic with only small windows of down time on the tracks to install the new superstructure. Because of the lengthy curing time required for cast-in-place concrete and the extensive falsework required for casting, this type of superstructure is seldom used these days.

Similar to the steel bridges, the concrete bridge designs consider a 12 foot wide by 8 inch deep concrete slab to support the ballast. Unlike the steel however, the slab in the concrete bridges can be cast monolithically with the beams and further can be considered to contribute to the strength of the superstructure. In all, four reinforced concrete superstructures were designed, one each for a 20', 30', 40' and 50' span.

The third type of conventional railroad bridge system considered utilizes precast, prestressed concrete girders. Similar to reinforced concrete, only four designs were considered for prestressed concrete, one each for a 20', 30', 40' and 50' span. In the case of prestressed concrete, a parametric study regarding the number of girders and span to depth ratios has been neglected due to the fact that typically there is limited variability in the cross-sectional dimensions of the standard precast products. The variation in the design to accommodate the strength limit states is compensated for in the number of prestressing strands used and the amount of mild reinforcing required for shear.

Prestressed concrete beams have evolved as one of the more predominant structural systems considered in both new and replacement structures for railroad bridges. This is particularly true for bridges approximately 30 feet in length, which comprises a large sector of the potential market. The prestressed bridge alternates developed for this study all utilize commercially available standard sections. All span ranges utilized four, 36-inch wide rectangular box sections per track. The advantages of these sections are that when placed side by side, as is generally required for railroad bridges, the tops of the boxes provide a wide flat surface suitable for supporting the track ballast without any additional concrete topping slab or deck pans.

4.1.3 Comparable Hybrid-Girder Designs

Having established a basis for comparison to conventional railroad bridge structures, the next task was to develop bridge designs for the hybrid-composite girders utilizing the same design criteria and dimensional parameters. In order to perform these initial design studies it was necessary to develop the methodology and equations necessary to quantify the limit states for design of the hybrid-girders. The development of the spreadsheet to evaluate the design limit states for the proposed beam will be discussed in more detail in the section on Preliminary Design & Analysis. Considerable parametric studies addressed the variability inherent in the components of these beams including dimensional variations as well as variations in the constitutive properties in the materials used to fabricate the beams. Designs using both carbon fiber and prestressing steel as the tension reinforcement have been considered.

The overall costs for the hybrid-composite girders can be broken down by the material costs, the labor for fabrication and the labor and equipment for erection. As with any construction product, the overall costs are impacted by the quantities required for a specific application. The costs of fabricating one girder will far exceed the costs of manufacturing hundreds of girders both due to material prices as well as tooling costs associated with economies of scale. For purposes of this investigation, it will be assumed that the tooling and equipment costs can be amortized over the production of one hundred girders of the same cross-section and length. This is similar to the amortization of tooling materials in the precast industry. Table 3 demonstrates a breakdown of the costs associated with the fabrication and erection of a typical hybrid-composite girder bridge of approximately 30 feet in length. The costs per linear foot of bridge are based on a 12-foot wide deck cross-section with eight beams that are 36 inches tall by 18 inches wide.

Component/Task	Unit	Quantity	Unit Cost	Cost/beam	Cost/(Track Ft.)
Material Costs					
Resin/Catalysts/Accelerators	**Lbs.**	141.1	$2.00	$282	$75.28
E-Glass Reinforcing	**Lbs.**	141.1	$1.80	$254	$67.76
Polyiso Foam Core	**Cu. Ft.**	116.3	$10.57	$1,229	$327.76
Conduit for compression reinf.	**Cu. Ft.**	18.8	$20.63	$387	$103.20
Compression Reinforcing	**Cu. Ft.**	18.8	$15.00	$281	$74.93
Tension Reinforcing	**Lbs.**	645.2	$1.50	$968	$258.13
Tooling (molds)	**Each**	1	$150.00	$150	$40.00
Vacuum Pump	**Each**	1	$3.00	$3	$.80
Misc. VARTM Materials	**Each**	1	$150.00	$150	$40.00
Subtotal Materials				**$3,704**	**$987.73**
Estimated Labor for Fab.	Man-hours	48	$55	$2,640	$704.00
Subtotal Fabricated				**$6,344**	**$1,692**
Shipping	**Cu. Ft.**	135	$3.00	$405	$108.00
Est. Labor for Erection	**Lump Sum**	1	$50	$50	$13.33
Erection Equipment	**Lump Sum**	1	$55	$55	$14.67
Subtotal Erection				**$510**	**$136.00**
Subtotal				**$6,854**	**$1,828**
Contingency (10%)				**$685**	**$183**
Total Cost – In Place				**$7,539**	**$2,010**

Table 3. Estimated Costs for Hybrid-Girders (30-foot Span)

4.1.4 Life Cycle Cost Analysis & Cost Comparisons

In order to evaluate the relative costs of the hybrid girder system compared to bridges constructed using conventional materials, sample designs have been compiled for 20, 30, 40 and 50 foot spans. As with the steel bridges, these designs assumed a non-composite concrete slab supporting the ballasted track work. The initial cost associated with a bridge superstructure comprised of these beams has been tabulated. These values are plotted on the graph shown in Figure 1 along with cost curves for conventional steel, concrete and precast bridges. To simplify the graph, only steel costs for plate girders with span to depth ratios of 10 are shown as these represent the more economical steel structures. It should be noted that the superstructure costs do not include the costs of the ballast and track work. Also note that the superstructure framing represents approximately 40 to 50% of the overall cost of a bridge replacement structure.

The cost curves on this graph confirm the logic in selecting precast concrete box beams over steel girders in the shorter span ranges. Although the cost data is limited to spans under 50-foot, it should be noted that there are some practical limitations to using precast concrete girders on much longer spans for railroad bridges. As a result, longer span railroad bridges are usually still constructed using steel girders. Regarding the hybrid-girders, the preliminary cost data developed indicates that as the span length increases, the hybrid-girders are less expensive than steel girders and the costs begin to approach those for precast box beams. It is also evident that for the hybrid-girders, a short span bridge can actually be more costly on a track foot cost than for a longer span. Both of these trends can be explained by a more in depth evaluation of the dimensional parameters of the beam.

Again the hypothesis for the economic success of this product has always been that the FRP components contribute substantially to the overall first cost and that by reducing the amount of FRP, the cost of the beam can become competitive with conventional construction methodology. Load sharing of the shear forces through the arching action of the compression reinforcing results in a thinner laminate for the webs. The stiffening of the webs resulting from the bond to the core also makes it possible to neglect elastic shear buckling and fully exploit the shear capacity of the glass webs. Experimental test results lend credibility to both of these behavioral characteristics. Parametric studies of the beam designs indicate that the same thickness of FRP in the webs that works for the 20-foot span, will also work for the 50-foot span. Given this and some practical limitations on the thickness of the glass fabric for the box beams, the indications are that the shorter span beams have a disproportionately greater amount of glass reinforcing than the longer span girders. Subsequently, the shorter span hybrid-composite girders also have a disproportionately higher cost.

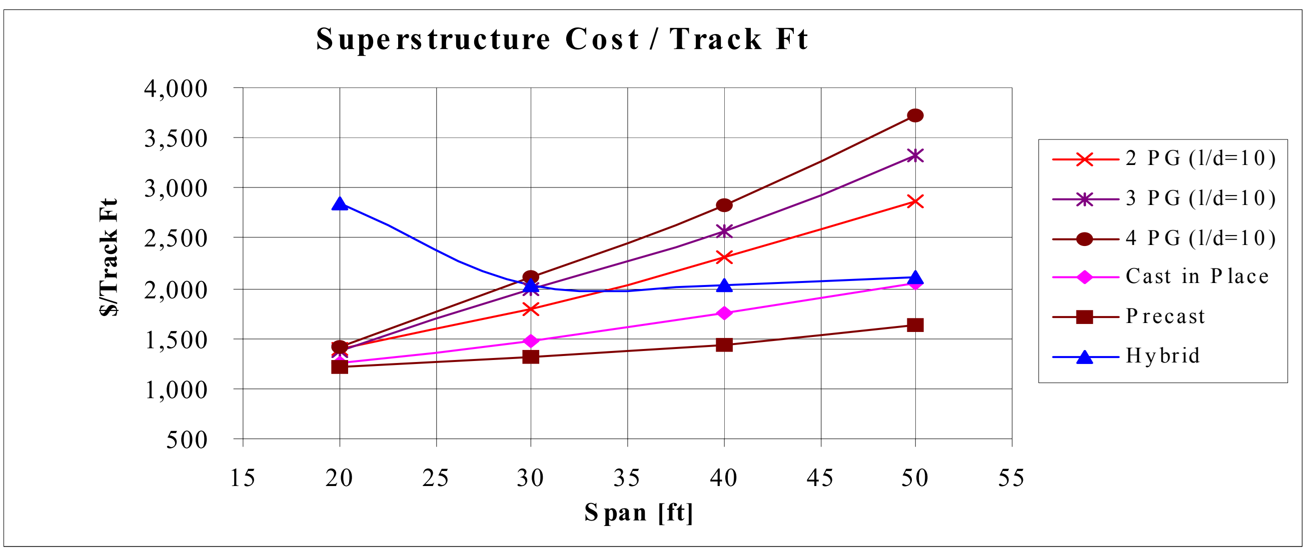


Figure 1. Cost Comparison of Hybrid-Girder Bridge vs. Conventional Materials

One of the primary benefits of the proposed hybrid-girder system is the resistance to corrosive environments inherent in FRP. This is in contrast to the susceptibility to corrosion in both steel structures and concrete structures. Although the hybrid-composite girder is a very efficient and cost effective embodiment of an FRP beam, the fact remains that it may not always be competitive with conventional bridge structures on a first cost basis. There is a growing trend in the transportation industry to recognize that there may be other inherent benefits to some materials that result in savings over the life of a structure that when evaluating all of the costs, can make a product with a higher initial cost seem more attractive. This is the basis behind a Life-Cycle Cost Analysis (LCCA).

Some of the costs that are generally included in LCCA include initial construction costs, operations, maintenance and repair (OM&R) and disposal costs. Initial construction costs are self-explanatory. O,M&R costs associated with railroad bridges can include, painting of steel structures, patching of concrete structures, reinforcing deteriorated girders and replacement of deck pans. The latter may also include the replacement of ballast, ties and rails. User costs, in the case of highway bridges can be quantified in terms of delays for private and commercial traffic, such as the personal time of the drivers as well as operating costs of the vehicles. Loss of revenue due to delays is one user cost associated with railroad bridge construction. Other costs include third party costs that are incurred by entities that are neither the agency nor the user. These costs are usually associated with the loss of revenue that may be incurred by a business adjacent to a construction site. For highway bridges empirical formulas and data exist to help quantify user and third party costs, however these costs are more difficult to ascertain for railroad bridges.

Finally, consideration can also be given to New Technology Introduction (NTI) costs [2]. These are costs associated with research, development and testing that may be incurred by an agency or some other entity. These are costs that ideally will be diluted or disappear over the course of general acceptance of the new technology, but that should be recognized in the LCCA.

Other major considerations in performing LCC analysis are the length of the analysis period, the target design lives of the alternates and what interest rates should be considered. For purposes of this investigation, the interest rate used will be the real discount rate of 3.9% and the inflation rate of 1.9% as published in the Office of Management and Budget, Circular No. A-94 [3]. This is the document that is to be used for benefit-cost analysis on all federally funded programs. In terms of design life, it is difficult to quantify design criteria that will ensure a prescribed service design life for a structure. The AASHTO LRFD Bridge Design Specification specifies a target life for highway bridges of 75 years. Although the AREMA code does not specify a target design life, historically railroad bridge designs are conservative both to provide an adequate factor of safety as well as minimize the maintenance requirements.

Given the variations that can exist in the numerous costs, interest rates, design lives, etc, it is difficult to generalize the life-cycle performance of for competing materials. In particular it is hard to quantify user, third party and NTI costs without having more information related to specific project or type of project. Regardless, a LCCA has been conducted as part of this investigation to compare a hybrid-girder bridge to conventional bridges of prestressed concrete and steel. This analysis has been simplified to include only construction costs and some minimal O,M&R costs. The analysis considers a typical 30-foot span for each material and includes substructure, superstructure and ballasted deck costs. The results are intended to demonstrate the improved life-cycle performance of the hybrid-composite girders assuming that the life expectancy of a bridge might be increased to 100 years as opposed to 75 years for a conventional structure.

Alternate	Initial Cost	% of Low Cost	Present Worth Cost	E.U. Annual Cost	% of Low Cost
Prestressed Concrete	$87,402	100%	$110,477	$4,309	100%
Steel Girder	$121,648	139%	$171,925	$6,705	156%
Hybrid Girder	$118,316	135%	$136,523	$5,324	124%

Table 4. Life Cycle Cost Analysis for Typical 30 foot Spans

The LCCA still indicates that concrete girders are more economical for a 30-foot span. However the Equivalent Uniform Annual Costs demonstrate that an increase in service life can result in the hybrid-girders having greater benefits over steel as well as being more competitive with concrete.

4.2 MATERIAL SELECTION PROCESS

4.2.1 FRP Shell – Matrix

In determining the specific materials for fabrication, the first component of the shell to consider is the FRP used to fabricate the webs and flanges of the girder. A detailed discussion regarding the numerous variations and combinations of materials that qualify as FRP is beyond the scope of this investigation. For simplicity, the definition of FRP for purposes of the hybrid-composite girder will be confined to a thermosetting or thermoplastic resin matrix reinforced with glass or carbon fibers. Even within this context there are several different types of resin that can be utilized for the matrix. Fabrication methods, as well as compatibility with the reinforcing fibers can influence the selection of an appropriate resin. Based on the use of glass fibers and the intended fabrication process, it was determined that a vinyl ester resin would be best suited for the matrix of the FRP.

4.2.2 FRP Shell – Glass Reinforcing

The second element required in the FRP is the reinforcing. This is the element that provides most of the strength and stiffness to the FRP component. In selecting the reinforcing it is necessary to consider both the material and orientation of the fibers. One of the inherent advantages of FRP is that optimization of components can be facilitated by orienting the fibers in the direction that best accommodates the strains induced in a structural element. This results in non-isotropic behavior, or more simply stated, the constitutive properties, i.e. strength and stiffness of the material can vary relative to the direction of the induced strains in an element. In general the strength and stiffness of an FRP component can be increased by positioning a larger percentage of the reinforcing fibers parallel to the direction of the larger anticipated strains due to applied loads. This is in contrast to structural steel, which is an elastic, perfectly-plastic, isotropic material that has exactly the same constitutive properties in all directions.

Although the orientation of the fibers provides greater flexibility with respect to design optimization, there can be some negative aspects to this attribute. Whereas the engineer has the luxury of custom tailoring the fiber orientations for optimum structural performance, this customization can result in an increase in fabrication costs due to the lack of standards. There are also additional costs associated with the engineering required to optimize the fiber layout. In many instances, optimization of the fiber orientations can be simplified by using standardized multi-directional woven glass fabrics. These woven glass fabrics are generally comprised of multiple layers of glass rovings with varying percentages of the fibers oriented in the 0^o, 90^o and $+/-45^o$ directions relative to the longitudinal axis. In some cases the fabric can be a "Unidirectional Weave" with all of the fibers oriented along the longitudinal axis or 0^o. In some cases it can be a "Biaxial Weave" with the same or different percentages in the 0^o and 90^o directions. In other cases the fabric can be a "Triaxial Weave" or a "Quad-Weave" with fibers oriented in multiple directions. These fabrics enhance the efficiency of a composite subjected to shear strains as well as longitudinal and transverse strains. Another benefit of the multi-directional weaves is that with the proper orientation of fibers, the composite begins to behave somewhat "quasi-isotropic" which can simplify preliminary designs.

Knowing the constitutive properties of the matrix and fibers, the constitutive properties of the overall FRP composites can be calculated to a reasonable level of accuracy using equations derived using the "rules of mixture". Under the "rules of mixture" theory, the properties of the matrix and fibers are combined in a sort of weighted average to determine the constitutive properties of the overall composite. In order to use these equations it is also necessary to know the fiber volume ratios of the manufactured composite, i.e. the volume of fibers with respect to the total volume of the composite. Although these equations are relatively accurate it is generally better and in some cases necessary to determine the strength and stiffness properties through experimental testing of coupon specimens.

The material selected for the glass reinforcing for the prototype testing is a "Quad-Weave" fabric known as QM6408, which is manufactured by Saint-Gobain BTI. The fiber distribution for this fabric is 45% in the 0^o direction, 25% in the $+/-45^o$ direction and 30% in the 90^o. The "Quad-Weave" was selected in part because the same fabric will be used in both the webs and the flanges. Subsequently, it is desirable to have fibers in all four directions to resist longitudinal and vertical strains as well as shear strains. The QM6408 was also selected based on past experience and the availability of test data to support the material properties.

4.2.3 FRP Shell – Core

Another significant component of the FRP Shell is the core material. The core material is intended to fill the interior volume of the beam shell with the exception of the voided space required for the compression reinforcing. The core material serves several purposes. One of the main functions is to provide the interior form that helps the beam maintain its shape during the fabrication process. Without the core, the other components of the beam would implode under the vacuum process. Another function of the core material is to provide lateral stability to the FRP webs of the beam shell. As mentioned, adhesive bonding of the FRP matrix to the core results in a subsequent increase in the elastic buckling strength of the FRP webs for both axial and shear strains. Lastly, the core material contributes to the strength of the overall hybrid-composite girder in two ways. The portion of the core material positioned above the compression reinforcing helps to distribute the vertical loads into the other components of the beam. The portion of the core below the compression reinforcing can also assist in resisting the shear forces between the compression reinforcing and the tension reinforcing.

Several different materials have been investigated as potential core materials. Some of the more promising materials investigated include Polyisocyanurate (Polyiso) Foam, Balsa Wood and Polyvinyl Chloride (PVC) Foam. Table 5 indicates the relative cost and material properties for each of these materials. For PVC Foam a range of costs have been shown to indicate how the density effects the mechanical properties and cost. All costs are based on one square foot of core material that is 1 inch thick, i.e. one board foot.

Core Type	Density (pcf)	Cost ($/Ft^2)	Compressive Strength (psi)	Compressive Modulus (psi)	Shear Strength (psi)	Shear Modulus (psi)
Polyiso Foam	2.8	$1.17	47	1215	35	366
Balsa Wood	9.7	$2.28	1842	594500	435	24070
PVC Foam (low)	4	$2.57	115	8700	100	3190
PVC Foam (high)	10	$18.21	500	33350	380	9570

Table 5. Comparison of FRP Shell Core Materials

It is evident from this data that the strength and stiffness properties for balsa are significantly better than for the other materials investigated. When evaluating the performance of the materials based on costs normalized for engineering properties, the balsa is even more appealing. Despite this the material selected for the prototype was the polyiso foam. The reasoning behind the selection is partly based on cost, i.e. using polyiso will reduce the cost of the core material by half. Other reasons for the selection are based on availability and handling and shaping as the polyiso comes in larger sections than the balsa. The other rational is that experimental data indicate the additional costs for balsa are not warranted despite the enhanced performance benefits.

4.2.4 FRP Shell – Conduit

One component of the shell, which is more critical to constructibility than to structural performance is the conduit used to form the void for the compression reinforcing. The primary function of the conduit is to maintain a voided space within the confines of the beam shell to accommodate the compression reinforcing. In the preferred embodiment, this space would remain as a void until after the girder has been erected, at which time the compression reinforcing would be injected into the conduit insitu.

In some ways this is the most complicated component to develop for the hybrid girder. To maintain the dimensions of the voided space for the compression reinforcing during fabrication, it is necessary for the conduit to maintain its shape under both the exothermic reaction of the resin curing process as well as the vacuum pressure of the molding process. Another constraint that is imposed on the conduit is the need for the cross sectional dimensions of the duct to conform to the requirements for compression reinforcing and still fit in with the overall dimensions of the beam shell. Materials used for this component must also be capable of withstanding the exothermal reaction resulting from the curing of the resin.

During the preliminary fabrication experiments, both corrugated steel duct and HDPE ducts were tested for compatibility with the manufacturing process. In both cases it was necessary to use ducts with a circular cross-section based on availability. Ultimately, it appears to be more desirable to utilize a rectangular duct that conforms to the cross-section of the beam shell. One of the more desirable solutions would be to manufacture the conduits out of HDPE to the cross-sectional dimensions required. Although the technology exists to fabricate this type of product, current manufacturing of HDPE ducts is limited almost exclusively to circular cross-sections. Other thermoplastics that have been given consideration for the conduit are polyurethane and polypropylene. Thermoplastics in general can be more sensitive to exothermal reactions. Another consideration is to manufacture conduits out of glass FRP similar to the beam shell itself.

It may also be possible to utilize a removable mandrel in place of a duct to form the conduit. In this scenario the mandrel, e.g. an inflatable mandrel could be used to maintain the shape of the conduit during fabrication and then removed in order to inject the compression reinforcing. Another new product was also identified that can function as a removable mandrel. The material is a water-soluble casting material developed specifically for composite tooling and is marketed under the trade name of "Aquacast" by Advance Ceramics Research. Aquacast is a plaster-based material that can be molded to virtually any shape. For application in the hybrid girder, the material would be used to fill the voided space within the interior foam core. Once dried, the material will prevent penetration of the resin into the interior space for the compression reinforcing. Once the resin has been infused and cured, the Aquacast can be flushed out with tap water leaving an internal conduit to accommodate the compression reinforcing. One of the more appealing attributes of this material is that it can be recycled. Although the material has a relatively high initial cost, the ability to recycle makes it possible to amortize the cost over a number of beam fabrications. This material offers great potential for flexibility in design while also being potentially the more cost-effective alternate for the conduit.

Finally it should be noted that although there are distinct weight advantages from placing the compression reinforcement after erection of the beam, it is also possible to cast the reinforcement in the lay-up of the girder prior to the resin infusion, eliminating the need for the conduit.

4.2.5 Compression Reinforcing

The material utilized for the compression reinforcement must have a high compressive strength and modulus, but does not necessarily require high tensile or shear strength. One material ideally suited for this component is portland cement concrete. Concrete has long been one of the most common building materials, mainly due to its high compressive strength, low cost and its flexibility in conforming to whatever shape is required for a given structure. In place of concrete, another logical choice for the compression reinforcing is the matrix of concrete, or portland cement grout. Depending on the aggregate, concrete typically has a slightly higher elastic modulus than pure grout. However portland cement grout, for this particular application, may prove more desirable with respect to the simplification of the pumping procedures that will be required to introduce the compression reinforcing insitu.

Consideration was also given to polymer concrete. Polymer concrete is generally similar in composition to portland cement concrete with one main difference. In polymer concrete, the hydraulic cement and water matrix used to hold the aggregates together is replaced with polymer compounds such as epoxy, latex or methyl methacrylates or (MMA). This can result in concrete with considerably higher compressive strengths than portland cement concrete, e.g. 10,000 to 12,000 psi versus 4,000 to 6,000 psi. Polymer concrete typically has considerably higher tensile strengths as well. However in addition to the cost premium for polymer concrete, the biggest drawback to this product as a potential for the compression reinforcing within the hybrid-girder is that the polymers used in the matrix tend to have a much lower elastic modulus than for a portland cement matrix. This results in concrete with a much lower compressive modulus, e.g. 1,100 ksi versus 4,000 ksi for portland cement.

Another commercially available product is a new type of reactive powder cement produced by LaFarge under the trade name "Ductal". Because of the proprietary nature of this product, limited information is available regarding the composition. However, the product is very similar to concrete in nature and utilizes either steel or poly-microfibers in conjunction with the matrix. The end product results in concrete with twice the elastic modulus and nearly seven times the compressive strength of portland cement concrete. Although there is a cost premium associated with this product, the performance characteristics make it very appealing for use as the compression reinforcing in the proposed hybrid-composite girder. Based on a review of the performance characteristics of Ductal, further research with this product is warranted.

4.2.6 Tension Reinforcing

The last major component is the tension reinforcing. Again the tension and compression reinforcing are the primary load carrying elements of the hybrid-composite girder. They also provide the greatest contribution to the elastic stiffness of the beam. Subsequently, the material selected for this component must have a very high tensile strength and a very high modulus of elasticity.

The primary materials that have been considered for this component are unidirectional glass fibers, unidirectional carbon fibers and conventional prestressing steel. Each of these three materials offers some distinct advantages over the others. For example glass fiber reinforcing has very high strength and being essentially the same material as that utilized for the beam shell, it offers some simplification in the lay-up for manufacturing. The down side of the glass fibers is that even using unidirectional rovings, which are more efficient than woven fabrics, the elastic modulus is relatively low. As a result, satisfying the serviceability requirements requires an excessive amount of the material above that required for strength.

The disadvantages of the low modulus associated with glass fibers can be improved upon dramatically with the use of carbon fibers. Carbon typically has much higher strength and stiffness properties as compared to glass. However, there is a cost premium associated with the use of this material. Further, the types of resins that are more compatible with carbon are not always the same as those that are compatible with glass. Regardless, carbon fibers still maintain a similarity in composition to the beam shell that results in a structural element without any materials susceptible to corrosive environments.

Another material, which provides a cost-effective solution, is steel conventionally used for prestressing of concrete. The most commonly available types of prestressing steel are 150 ksi deformed or threaded bars, or 270 ksi, seven wire strand. Both types of prestressing steel have an elastic modulus around 28,000 to 29,000 ksi, which is two to three times that of carbon fibers. There still appears to be no other material that offers the same type of performance characteristics as steel, even at a significantly greater cost premium. The down side of using this material is that the anchorage of the tendons is slightly more complicated than with glass or carbon fibers. The tensile strength is not as high as carbon either, however as stiffness appears to govern design the lower strength of the steel compared to carbon is not a detriment. The other disadvantages of steel are that it weighs more than four times as much as the glass or carbon and is sensitive to corrosive environments. These attributes are not considered as critical, since the amount of steel is relatively small and does not pose a dramatically higher weight for the beam. Regarding, the corrosive nature of the steel, this attribute is circumvented by the fact that the steel will be completely encased within the other components of the hybrid girder in a manner that will minimize the susceptibility to atmospheric corrosion.

Table 6 contains a matrix indicating the relative cost and performance data for the materials considered for the tension reinforcing component. The properties for glass and carbon are based on an FRP composite with an assumed 50% fiber volume ratio. It is evident from this matrix that the cost/lb. of carbon and glass fibers is very high compared to steel. In order to make a more rational evaluation, the cost data has been normalized to steel, assuming that designs for all materials will be stiffness driven and that strength will not govern. This normalization reflects the cost of providing a stiffness equivalent to steel, using the ratios of the densities, elastic moduli and the cubed ratio of the moment arms “d”, of glass and carbon relative to steel.

Material	Density (pcf)	Tensile Strength (ksi)	Elastic Modulus (ksi)	Cost ($/lbs)	Moment arm d (in)	Normalized Cost /steel $
Glass Fibers	120	120	4,800	$4.50	33.25	$3.30
Carbon Fibers	97	250	17,000	$16.00	33.25	$2.69
Prestressing Steel	490	150-270	28,000	$1.50	30.50	$1.00

Table 6. Comparison of Potential Tension Reinforcing Materials

It should be noted that by normalizing the costs based on the characteristics of the materials, it appears evident that carbon fibers can be more economical than glass fibers, even though the unit cost per weight is over three times that of glass. It is also important to recognize that although carbon is approximately 269% more expensive than prestressing steel, the tension reinforcing component only comprises roughly 10% of the total cost of the fabricated beam in place. Subsequently, utilization of carbon fibers in place of prestressing steel may result in less than a 25% increase in the overall cost of the beam and an even smaller increase in the overall cost of a bridge. Regardless, based on the evaluation of these materials, the material of choice for the prototype test specimen was prestressing steel.

4.3 PRELIMINARY DESIGN & ANALYSIS

4.3.1 Overview of Design & Analysis

In order to assess the limit states that are required to quantify the behavior of the hybrid-composite beam system, it is worth briefly considering the general evolution of structures through the course of time. Some of the first structures ever contrived were stone arches. Without any calculations at all, it became evident that by placing blocks of stone sequentially in a circular curve, it was possible to create a structure that would span a distance equal to the diameter of the circle. Centuries later, with the advent of materials such as iron and steel, it became possible to span greater distances with much lighter structures. As the steel rolling mills and fabrication technology became more sophisticated, complex riveted trusses gave way to sleek rolled sections and welded plate girders.

Engineers continued to experiment with hybrid structures, utilizing structural members comprised of combinations of building materials acting compositely to resist the applied forces. One of the most simplistic concepts in the history of structural engineering, which is now completely taken for granted, is the concept of adding reinforcing steel to a concrete beam to dramatically increase the load carrying capacity of the member. Nowadays, we simply refer to these as concrete beams. However in actuality they are really hybrid-composite structures, relying on a more optimized utilization of two very different materials.

In almost every instance, as engineers create new structural forms, it is necessary to develop a methodology for quantifying the behavior of the various materials within the specific embodiment intended. Such is the case with the evolution of fiber reinforced plastics (FRP). In general, various methods of structural analysis that evolved for other types of materials can also be applied to the analysis and design of FRP structures. Certainly statics is applicable. The theory of elasticity and general mechanics of materials are also applicable.

There are two fundamental assumptions that differentiate FRP structures from steel structures. One difference is that steel is assumed to be elastic, perfectly plastic. In other words, the stress strain relationship for this material can be accurately represented with a bilinear curve. This property also allows the steel to undergo a significant increase in strain after reaching its yield strength and prior to a brittle rupture. The other property of steel that grossly simplifies analysis and design is that it is an isotropic material. In other words, it has essentially the same constitutive properties in all directions.

In contrast to steel, FRP materials generally remain linearly elastic up to the point of a brittle failure mode. To further complicate analysis, FRP structural components are anisotropic rather than isotropic. In other words, the constitutive properties of these materials can vary in each direction, and are a function of the specific composition of the material. In the case of glass fiber reinforced composites and carbon fiber reinforced composites, the strength and stiffness properties are typically a function of the fiber volume ratios and the specific orientation of the fibers. Because of these fundamental differences, FRP structures require additional consideration in design and analysis. An in depth derivation of the theory of elasticity for non-linear, non-isotropic material behavior is beyond the scope of this project. However, this information is available in numerous texts and will be elaborated on where it is necessary to evaluate certain limit states. One generalization that can be made is that, based on the development to date, it is not evident that a rigorous analysis of the FRP materials is warranted to arrive at a functional design.

4.3.2 Spreadsheet Development

By definition, the proposed hybrid-composite beam is comprised of several different materials. Each component of the beam is well suited to satisfying one or more design limit states. The materials are arranged in such a manner as to optimize the structural behavior of the overall beam. With regard to some limit states, the hybrid-beam behaves similar to a reinforced concrete beam. In some respects, it behaves similar to a steel box beam. In others it behaves like a tied arch. Finally, with respect to many of the limit states, the FRP components do contribute to the resistance. As a result, some additional consideration has to be given to the analysis of these components of the structure.

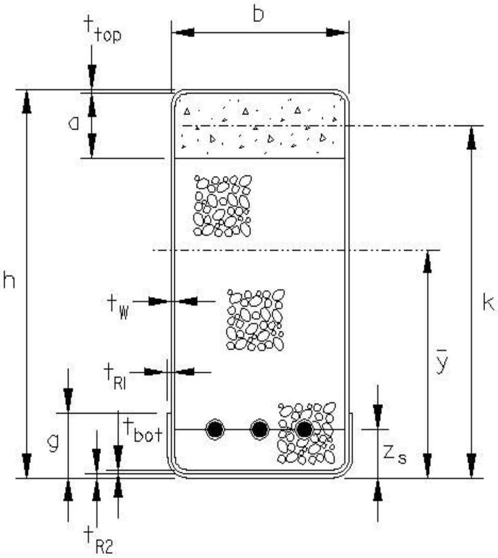
One of the primary goals in developing an efficient design for any structure, is to determine a predictable mode of failure and attempt to provide and optimized design for all of the limit states such that the intended failure mode will still govern. It was not evident at the start of this research project, how each limit state would be quantified, or how the behavior of each different component of the beam would influence the design. Because of this lack of intuitive knowledge, a design spreadsheet was initiated at the beginning of the project that would provide a means of investigating the limit states simultaneously.

The following sections of this report will provide a more detailed discussion of the more critical design limit states considered in the development of the hybrid-girder system. In general one worksheet within the overall spreadsheet has been added to address each specific limit state. The following is a list of the sheets within the spreadsheet:

- SECTION
- DESIGN LOADS
- SHEAR
- DEFLECTIONS
- BENDING-1
- BENDING-2
- WEB CRIPPLING
- Cooper E-80
- HS 20-44
- Sections IO
- Cooper E80 Alt.
- Cooper E80 Locos

The last five worksheets included in the spreadsheet, including *Cooper E-80* through *Cooper E80 Locos* are strictly devoted to calculation of design forces. The worksheet designated *Sections IO* contains a live load generator to obtain moment and shear envelopes used to evaluate specific limit states.

The first worksheet was set up to define the cross-section of the hybrid-composite girder and the constitutive properties of various materials used. Figure 2. demonstrates a generalized beam cross-section showing both steel tension reinforcing and carbon fiber tension reinforcing. The variables used to describe the cross-section are shown below.



Dimensional Variables

h = overall depth of beam shell
a = depth of compression reinforcing
b = overall width of beam shell
g = vertical height of carbon reinforcing on sides of beam
k = bottom of beam to centroid of compression reinforcing
t_{top} = thickness of top FRP flange
t_{bot} = thickness of bottom FRP flange
t_{R2} = thickness of carbon tension reinforcing on bottom flange
t_{R1} = thickness of carbon tension reinforcing on sides of beam
t_w = thickness of FRP beam shell webs
A_s = area of steel tension reinforcing
z_s = bottom of beam to centroid of steel tension reinforcing

Figure 2. Typical Cross-Section Geometry

In calculating the section properties of the hybrid-composite beam, it is necessary to consider the relative constitutive properties of the various materials used in order to simplify analysis. As a result it is necessary to select the elastic modulus of one material to serve as the basic value to be used in analysis. In the case of the hybrid-composite beam, it was determined that a logical choice for the base material would be the glass FRP webs. The primary reason being that this is the material most likely to remain relatively constant whereas the material selected for the compression and tension reinforcing could result in dramatically different elastic moduli. Further, the elastic modulus of concrete is not well suited as a constant, in that concrete exhibits non-linear material behavior as it approaches its ultimate strain. By selecting one material as the reference, all of the other materials comprising the beam are then transformed into equivalent areas of this same material using the respective modular ratios "n". As a result, the modular ratio used for transforming steel tension reinforcing to an equivalent amount of FRP would be $n_s=E_{steel}/E_{web}$ and likewise for the other materials. The non-isotropic FRP laminate properties are quantified by elastic moduli, shear moduli and Poisson's ratios in the two orthogonal directions. The longitudinal elastic modulus of the FRP is used to calculate the transformed section properties for bending.

Once all of the geometric data and constitutive properties of the materials have been input, the program calculates the cross-sectional area and moment of inertia for the beam under consideration at tenth points along the span. The self-weight of the structural member is also calculated, both with and without the compression reinforcing in place. The calculated properties of the section are then used in subsequent worksheets to help design the components of the beam for the various design limit states.

The second worksheet is dedicated to compiling the governing forces that will be used in designing the beams. The spreadsheet has been generalized to allow the calculation of the design forces in accordance with AREMA for railroads, AASHTO for highway bridges as well as other codes specifically for building applications. The following is a brief description of the book keeping procedures necessary to compile the design forces and allow the designer to tailor the calculations to a specific application.

- ***Bridge Geometrics and Composition:*** The designer must define the cross section of the overall structure to be considered. In defining the cross-section, the designer must identify all of the superimposed dead loads to be applied to the cross-section. Some of the information in this section might include; span length, deck width, depth of ballast, slab thickness, parapet loads, overlay thickness and weight and rail loads.

- ***Dead Loads:*** In this section, the designer must select the number of girders to be included in the overall cross-section of the structure as well as the girder spacing. Based on this information and the previously calculated cross-sectional properties, the program calculates the dead loads per track foot and subsequently per foot of beam.

- ***Superimposed Dead Loads:*** In many cases, portions of the dead loads on a structure may be placed or removed at a later time within the life of the structure. As a result, it is usually beneficial to track these loads separately than the basic dead loads. Superimposed dead loads typically include items such as parapets, overlays, ballast, or in the case of buildings it could include items such as interior partitions.

- ***Live Loads:*** The information provided previously, including the number of girders and the beam spacing are used by the program to calculate the distribution of live load to the specific beam under consideration. The span length previously entered is used to determine the impact factors to be applied to live loads for highway and railroad bridges.

  The program is set up to obtain the live load design forces from look up tables of governing loads based on AREMA and AASHTO. A live load generator for moving loads was also built into the spreadsheet to facilitate getting corresponding moments and shears at specific sections for shear calculations.

- ***Maximum Design Forces:*** Once all of the dead, superimposed dead and live load forces have been determined for a specific beam, the program combines them to calculate a single force for the limit states of shear and bending. The unfactored loads are considered service loads. The program also takes into account combinations of the forces using the appropriate load factors to be considered in ultimate strength design.

There are various different classifications of limit states that are applicable to structural analysis and design. This investigation considers two types of limit states, strength and serviceability. Strength limit states are typically those that must be satisfied in order to ensure the safe performance of a structure. Serviceability limit states are those that are generally evaluated based on some subjective criteria relative to an acceptable level of performance, as determined by the user, relative to a specific behavior of the structure, e.g. deflections or vibrations. In general, as structural optimization evolves with respect to strength limit states, the criteria to satisfy serviceability limit states becomes more prevalent in design. This is often typical of lighter weight structures of any material and has also been evident through the development of FRP structures utilized in civil applications.

While it is generally preferred to select either a service load, working stress (ASD) or a factored load, factored resistance (LRFD) approach to design, in some cases there is a duplicative evaluation of a design limit state using both philosophies. One instance of this is in the design of prestressed concrete beams where it is common to design according to some pre-determined allowable stresses for the materials (ASD), but still check the ultimate capacity of the beam using LRFD. It will be evident in subsequent sections of this report, that for some limit states, it was desirable to mix design philosophies in order to quantify the design. The rationale will be dealt with in the appropriate sections of the report.

4.3.3 Deflection Limit State

Deflection of a structure is a serviceability limit state that usually has to be satisfied, regardless of the intended use of the structure. The major difficulty in satisfying the deflection limit state with FRP structures, is that although these materials have excellent strength characteristics, they still have a relatively low elastic modulus. As a result, the design of these structures for civil applications generally requires an excessive amount of material over what is required to meet the strength limit states, and all economy is lost. The embodiment of the hybrid-composite beam uses less expensive materials such as concrete, steel and/or an efficient application of carbon fibers to meet the deflection limit state. Preliminary analysis and testing indicate that serviceability still governs design of the hybrid-composite beam system, but the over-design for strength is substantially reduced over that of a homogenous FRP beam, resulting in a more economical structure.

It is important to note that in many conventional bridge structures the cross-sections of the girders are prismatic, or constant over the length of the beam. For simply supported beams, this grossly simplifies deflection calculations. Where cross-sectional properties vary over the length, the assumption of a constant flexural rigidity (EI) over the length is no longer valid. Variation in the rigidity over the length of the beam has to be taken into account, even for simply supported beams. Such is the case with the hybrid-composite girder.

In order to account for the variable location of the compression strut along the length of the beam, the spreadsheet is set up to calculate the sections properties at 1/10 points. The Moment Area Theorem is used to calculate the live load deflection of the beam at mid-span using numerical integration of the moment diagram and the variable section properties. In order to simplify the calculations and facilitate a quicker evaluation of the deflection limit state, the live load moment diagram for a simply supported structure can be accurately approximated be deriving an equivalent uniform live load for the Cooper E-80 locomotive. This equivalent uniform load will produce the same maximum moment and subsequently an equivalent amount of curvature in the beam.

The deflection criteria for specific applications addressed in the design spreadsheet come from the various design codes previously mentioned. The allowable deflection is typically a function of the span length "L". The values shown below assume the span length "L" is represented in units of inches.

- L/360 – Typical for building structures
- L/800 – AASHTO criteria for bridges
- L/1000 – AASHTO criteria for bridges with pedestrian sidewalks
- L/640 – AREMA criteria for steel and prestressed concrete railway bridges

In evaluating structures for live load deflections, only service level (unfactored) live loads should be considered in the deflection calculation. This is the case regardless of whether the design is being conducted using WSD or LRFD. In many cases it is still necessary to include the impact factors applied to the live loads, which are different from load factors.

One final observation regarding deflections is that a beam with a prismatic cross-section, having a constant moment of inertia equal to the mid-span value of the hybrid-composite beam, will have live load deflections that are approximately 7/8 those of the hybrid-composite beam with a variable moment of inertia. Subsequently, it could be said that there is some loss of efficiency with respect to the deflection criteria due to the profile of the compression reinforcement. However, as will be evident later, this slight reduction in stiffness is more than offset by a reduction in the web thickness resulting from the arching action of the compression strut.

4.3.4 Bending Limit State

Bending failure is one of the more critical strength limit states. For the hybrid-composite girder, the bending behavior at the ultimate limit state is analogous to a reinforced concrete beam in many ways. In evaluating the ultimate strength design of reinforced concrete it is assumed that the concrete below the neutral axis has cracked, is in tension and no longer contributes to the strength of the beam. Plane sections are assumed to remain plane, but the stresses in the concrete are not linearly proportional to strain at ultimate.

For simplification in calculations, it is assumed that at the ultimate strain, the concrete stress remains constant over the entire depth in compression. The magnitude of this constant stress is assumed to be some portion of the ultimate compressive stress in the concrete. The most commonly used value in the United States is the limiting value of $0.85f_c'$, where f_c' is the strength of the concrete determined from test cylinders. This equivalent stress is then applied over a depth, "a" which is some portion of the total depth of concrete above the neutral axis. This limiting stress applied over the specified depth results in a compression force that is equivalent to what would be found from a rigorous integration of the actual stresses in the concrete loaded in compression. Although there are several models that have been derived for this purpose, the one described above is commonly known as "Whitney's Equivalent Stress Block", and can be found in most reinforced concrete textbooks.

By assuming the compressive stress in the concrete at failure, it is possible to ascertain the amount of tension reinforcing required for a concrete beam by equilibrating the tension force with the compression force in the concrete. A balanced design and subsequently a ductile failure mode are assured in reinforced concrete beams by limiting the amount of the reinforcing steel such that the steel will yield prior to crushing of the concrete. When the failure mode becomes crushing of the concrete, the beam is considered to be over reinforced.

In its most simplistic form, one might consider the hybrid-composite beam without the FRP shell, in which case it would resemble almost exactly, the behavior of a reinforced concrete beam in bending, where all of the concrete below the neutral axis has been removed. However in the case of the hybrid-composite beam it is desirable to consider the strength contributions from all of the components. There are some other subtle differences as well. As previously mentioned the FRP material used in the beam will remain linear elastic to failure. Further, as mentioned before, the amount of tension reinforcing required is typically governed by satisfying the deflection criteria. As a result, whether steel or carbon is utilized for the tension reinforcing, it is likely that all of the materials except the compression reinforcement will remain in the elastic region. As a result, at ultimate bending capacity, the failure mode for the hybrid-composite girder is likely to be crushing of the compression reinforcement rather than a ductile failure of the tension reinforcement. Noting these differences, it is now possible to describe the methodology in calculating the ultimate bending capacity of the hybrid-composite beam.

Similar to a reinforced concrete beam, it is assumed that the concrete will reach its ultimate strain at the failure limit state for bending. For purposes of this investigation the value for ultimate strain in the concrete is assumed to be ε_{cu}=0.003. It is also assumed that plane sections remain plane and perpendicular to the neutral axis. The glass, carbon and steel reinforcing are all assumed to remain in the linear elastic range. Steel reinforcing does not remain linear elastic after yield. However due to the amount of steel necessary to satisfy serviceability, it is anticipated that the steel will not yield. The stress in the steel at bending failure is checked in the design spreadsheet to confirm this assumption.

Based on these assumptions it is possible to ascertain the strain in each component of the beam using strain compatibility. The various components of the hybrid-composite girder are identified by the following subscripts:

TF = Top Flange (Glass Reinforcing)
WT = Top portion of Web located above the neutral axis (Glass Reinforcing)
WB = Bottom portion of Web located below the neutral axis (Glass Reinforcing)
BF = Bottom Flange (Glass Reinforcing)
R2 = Bottom Flange (Carbon Reinforcing)
R0 = Bottom Side (Carbon Reinforcing – portion subjected to uniform stress)
R1 = Bottom Side (Carbon Reinforcing – portion subjected to triangular stress distribution)
S = Steel Tension Reinforcing
C = Compression Reinforcing

It should be noted that in most cases, the hybrid-composite girder will not necessarily have every one of these components, e.g. if steel tension reinforcing is used, it is not likely that the beam will also contain carbon fiber reinforcing. Figure 3 depicts the strains and corresponding stress distributions for the various components of the hybrid-composite girder. The equations for determining the moment arms "d" about the neutral axis and the strains in the various components are listed below:

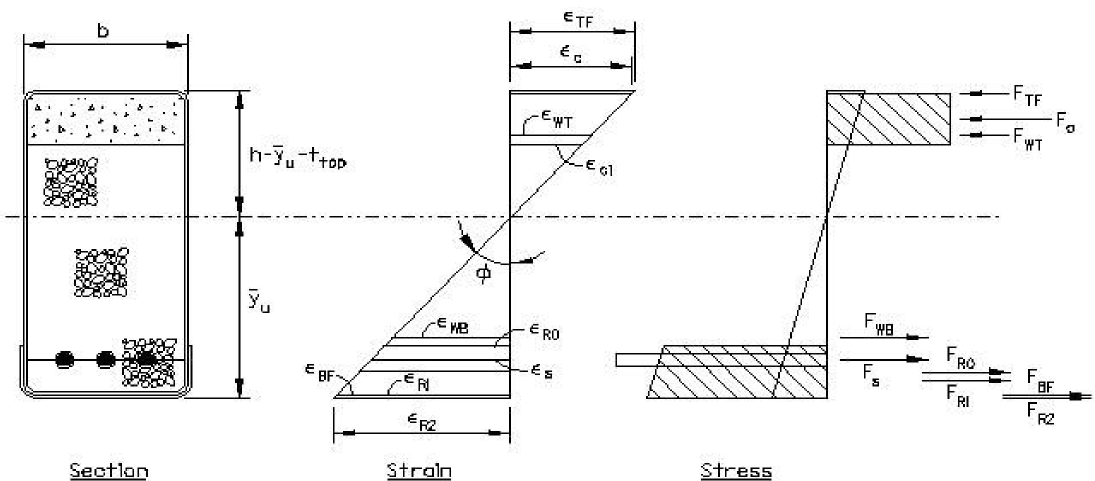


Figure 3. Strain and Stress Distribution and Resultant Forces on Section

$$\varepsilon_{TF} = \varepsilon_c \frac{h-\bar{y}_u}{h-t_{top}-\bar{y}_u}$$

$$\varepsilon_{WT}=\varepsilon_c \frac{h-\bar{y}_u}{2\left(h-t_{top}-\bar{y}_u\right)}$$

$$\varepsilon_{WB}=\varepsilon_c \frac{\frac{1}{2}\left(t_{R2}-\bar{y}_u\right)}{h-t_{top}-\bar{y}_u}$$

$$\varepsilon_{BF}=\varepsilon_c \frac{t_{R2}-\bar{y}_u}{h-t_{top}-\bar{y}_u}$$

$$\varepsilon_{R2}=\varepsilon_c \frac{\bar{y}_u}{h-t_{top}-\bar{y}_u}$$

$$\varepsilon_{RO}=\varepsilon_c \frac{g-\bar{y}_u}{h-t_{top}-\bar{y}_u}$$

$$\varepsilon_{R1}=\varepsilon_c \frac{g/2-\bar{y}_u}{h-t_{top}-\bar{y}_u}$$

$$\varepsilon_s = \varepsilon_c \frac{z_s-\bar{y}_u}{h-t_{top}-\bar{y}_u}$$

$$d_{TF} = -\left(h-\bar{y}_u - t_{top}/2\right)$$

$$d_{WT} = -\tfrac{2}{3}\left(h-\bar{y}_u\right)$$

$$d_{WB} = \tfrac{2}{3}\left(\bar{y}_u - t_{R2}\right)$$

$$d_{BF} = \bar{y}_u - t_{R2} - t_{bottom}/2$$

$$d_{R2} = \bar{y}_u - t_{R2}/2$$

$$d_{R0} = \bar{y}_u - g/2$$

$$d_{R1} = \bar{y}_u - g/3$$

$$d_S = \bar{y}_u - z_S$$

where $\bar{y}_u$ = position of the plastic neutral axis, PNA with respect to the bottom of the beam

It should be noted that the equations above do not assume that the concrete is at the ultimate strain of ε_{cu}=0.003, although they are valid for this assumption. These equations were generalized using ε_c instead of ε_{cu} assuming that using a rigorous calculation for the force in the concrete, it could be possible to use a material for the compression reinforcing that might not be at its ultimate strain when a bending limit state is reached. This formulation also facilitates tracing the moment rotation curve of the cross-section through the full range of loading.

The stress in each component can be calculated by multiplying the strain by the respective elastic modulus (E) for each component, i.e. $\sigma=E\varepsilon$. The corresponding force in each component is then determined by multiplying the stress times the respective area of each component, i.e. $F=\sigma A$. This is true with the exception of the compression reinforcing. As stated earlier, it is assumed that the stress in the concrete will be $0.85f_c$', and the force in the compression reinforcing will be $0.85f_c$'ab. Before calculating the actual strains, it is first necessary to calculate the plastic neutral axis or PNA. By taking the summation of the internal, horizontal forces in the beam, it is possible to solve directly for the PNA using the equation shown below.

$$\bar{y}_u = \frac{\left[bt_{top}h + t_w h^2 + \dfrac{0.85f_c' ab\left(h-t_{top}\right)}{E_w \varepsilon_c} + t_w t_{R2}^2 + bt_{bot}t_{R2} + n_R t_{R1} g^2 + n_S A_S z_S\right]}{\left[bt_{top} + 2t_w h + \dfrac{0.85f_c' ab}{E_w \varepsilon_c} - 2t_w t_{R2} + bt_{bot} + n_R bt_{R2} + 2n_R t_{R1} g + n_S A_S\right]}$$

Once the PNA is located, this value can be substituted back into the calculations for strain and the moment arms, "d_i" to calculate the nominal resistance of the cross-section. In order to simplify calculations, another method of arriving at the plastic neutral axis was derived using transformed section properties. This method involves calculating an elastic neutral axis in the same manner that is used to quantify the flexural rigidity for the deflection checks. The only difference is that the transformed section properties for concrete use a secant modulus to represent the reduction in compressive stiffness of this material as it is loaded in compression. The secant modulus, E_{sec} shown below is used in place of the elastic modulus of concrete E_c.

$$E_{sec} = \frac{E_c}{1+\left(\varepsilon_c/\varepsilon_{cu}\right)^2}$$

The method of using the transformed section properties with the secant modulus still satisfies internal force equilibrium. It was also found to yield results for the nominal moment capacity that are slightly conservative but still within 2% to 5% of the nominal moment capacity calculated using strain compatibility. This alternate method may also lend itself to simplification of the design equations for the bending limit state that is more conducive to hand calculations.

As yet a third check to quantify the ultimate bending capacity of the hybrid-composite beam, worksheet BENDING-2 was established to derive a comprehensive methodology for arriving at the plastic neutral axis and to quantify the actual force in the concrete. The calculations for this approach use the same strain compatibility approach as for the first method. The difference lies in the evaluation of the force in the compression reinforcing. Instead of assuming a uniform stress in the compression block, the actual stress relative to the strain in the concrete is used based on either a parabolic or a "Todeschini" stress-strain curve for concrete [3]. These stresses are then integrated over the height of the stress block to determine a more precise force in the compression reinforcing.

It should be noted that unlike a reinforced concrete beam, the stress block for the compression reinforcing will not necessarily begin at the plastic neutral axis. In fact it is undesirable to have the compression reinforcing extend below the PNA in that the materials used generally do not have any tension capacity. As a result, the range of the strains in the compression reinforcing will not necessarily be 0 to 0.003. Instead the strain at the lowest point in the stress block could still be relatively high. Using the closed form integration for the stresses in the compression reinforcing results in an average stress that is greater than $0.85f_c'$. The average values of stress found using this method were generally on the order of $0.95f_c'$. Arguably, as the compressive stress block moves away from the PNA, the average stress in the concrete should be approaching $1.0\ f_c'$.

The nominal moment capacity from the exact integration was higher than that for the other two calculation methods. However by using the average concrete stress derived from the closed form integration in place of $0.85f_c'$, all three methods of calculating the nominal moment capacity yielded almost exactly the same result.

A few final comments regarding the bending capacity check are warranted. The live load forces within the design spreadsheet for the ultimate bending check are calculated using the look-up tables for equivalent uniform loads to produce the maximum moment at mid-span. The load factors and load combinations used are from Chapter 8 in the AREMA code. Although load and resistance factor design (LRFD) is considered to be a very rational methodology for determining the safety factors in design, many codes still use load factors that were derived many years ago. In some cases these load and resistance factors have evolved over the years and may not be derived from a reliability analysis for the specific limit states and materials being considered. The primary reason for mentioning this is that the load factors assumed for this investigation merely represent very conservative values that are accepted within the railroad industry for design of concrete structures. Further evaluation of the hybrid-composite beam using an ultimate strength approach should also include an evaluation of rational load and resistance factors for the materials comprising the beam. Regardless, based on the factors in this investigation and the fact that, deflections generally control, the demand to capacity ratio for the bending limit state was generally found to be on the order of 0.5 to 0.6.

4.3.5 Web Shear Limit State

Quantifying the behavior of webs in a beam always presents some interesting challenges. Depending on the location within the beam, the webs can be subject to in-plane stresses both horizontally and vertically as well as shear stresses and out of plane bending stresses resulting from bending at the interface with the flanges. All of these components can be present in the webs of the hybrid-composite beam. To make things more interesting, the glass webs of the hybrid-composite beam are comprised of an anisotropic material. Web shear is also however, the limit state that derives the greatest advantage from the profiled shape of the compression reinforcing.

If the beam was comprised merely of a rectangular, glass reinforced plastic box beam, the webs would be unsupported over there entire vertical height. Intermediate vertical stiffeners could be spaced longitudinally along the beam. Regardless, the shear capacity of the material comprising the webs would be limited to the lesser of the allowable shear stress for the FRP based on some limiting strength criteria or the elastic buckling limit of the webs acting as a plate loaded in pure shear. In preliminary analysis, the shear resistance was evaluated both ways. The allowable shear stress for the material assumed in the design of the webs for the prototype beam is 7.5 ksi. This value was selected based on experimental determination of the shear strength of a quad-weave fabric (QM6408) with a vinyl ester resin [4]. A design material reduction factor of 0.5 has been applied, translating into a factor of safety of 2 against the service load demand.

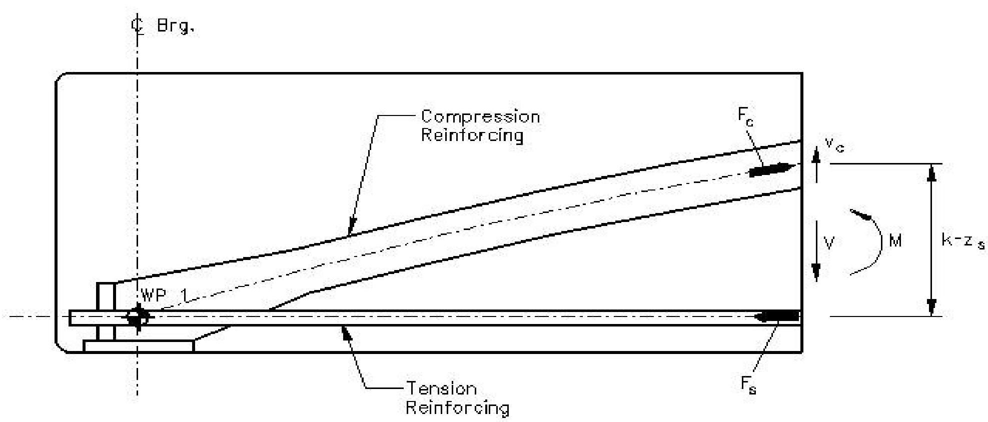
Due to the anisotropic nature of the FRP, calculation of the elastic buckling strength is somewhat more complex. However it can be calculated using the equations for the elastic buckling of orthotropic plates that can be found in several texts on advanced theory of elasticity [5,6].

Similar to a steel beam, it is assumed that on the demand side, the shear stress in the webs at any given location along the beam is equal to the corresponding shear force at that section divided by the area of the webs, i.e. τ_w=2t_wh. As with the bending limit state, we still adhere to the assumptions that plane sections remain plane and normal to the neutral axis and stresses are proportional to strains.

The determining factor over whether shear strength or elastic buckling controls the shear limit state is the bond between the core component and the glass FRP webs. Based on the experimental testing, it was found that the webs are stiffened by a resin bond to both the compression reinforcing and the polyiso foam core on the interior of the shell. It was found that this bond resulted in sufficient bracing of the webs to prevent the occurrence of elastic buckling prior to reaching the allowable shear strength for the material.

In evaluating shear further consideration also needs to be given to the reduction of the shear force in the webs resulting from the component of shear force taken in the compression reinforcing. The shear behavior of the hybrid-composite girder is somewhat analogous to a simply supported truss with a curved top chord. If the top chord follows a parabolic profile and the truss is subjected to a uniform symmetrical load, then the diagonal members between the top and bottom chord could be removed and the chords would be subjected to purely axial loads. This analogy also assumes that elastic buckling of the top chord is not a concern. For this same truss subjected to unsymmetrical loads, a significant portion of shear is taken in compression in the inclined top chord. However, the remaining portion of the shear force on a section has to be taken in webs or diagonal truss elements.

The diagram shown in Figure 4 demonstrates evaluating the reduction in the shear force in the webs of the hybrid-composite girder. This figure shows the internal forces acting on the compression and tension reinforcing. The component of shear taken by the compression reinforcing is V_c. The magnitude of V_c is calculated by dividing the moment at this section by the moment arm (k-z_s) to get the axial force in the tension reinforcing, which is equal to the horizontal component of the force in the compression reinforcing. Knowing the slope of the compression reinforcing at the given location, it is then possible to calculate the vertical component, V_c, of the resultant compression force. As the profile of the compression reinforcing for this investigation has been established as a continuous parabolic arch, it is easy to calculate the slope of the compression reinforcing at any given point by simply taking the first derivative of the continuous function for the profile of the arch. The equation for calculating the net shear on a section then becomes:



$$V_{net} = V - M\frac{k'}{(k - z_s)}$$

Figure 4. Internal and External Forces at a Section Along the Beam

In this equation (k') is the slope of the profile at the given location. In the event that carbon reinforcing along the bottom slab is used in place of steel, the z_s term is left out. It is important to note that due to the nature of moving loads, an accurate calculation for the net shear requires that the moment used in this equation be calculated for the same position of the live load that is used for calculating the shear, i.e. corresponding moments and shears. This is accomplished in the spreadsheet by using a moving load generator that examines all sections along the length of the beam.

By taking advantage of this load sharing effect resulting from the arching action of the compression reinforcing, the net shear in the FRP webs can be reduced substantially. For the prototype test specimen subjected to a Cooper E-80 moving load, the net shear was calculated to be approximately 37% of the total shear for the critical sections at the ends of the beams. In essence, as a result of this behavior, the net shear in the webs becomes nearly constant. This can result in a substantial decrease in the size of the FRP webs required.

4.3.6 Web Crippling Limit State

One of the primary functions of the portion of the webs above the compression reinforcing is to transfer the vertical loads from the top flange down to compression strut similar to a spandrel arch. As mentioned previously, this portion of the webs is also subjected to other components of stress and the analysis to determine the stress tensors requires an account of variations in constitutive properties of materials in different directions, many of which are coupled.

The web crippling phenomenon in steel beams is also complex and subject to many of the same stress components. Again analysis of steel structures is simplified by the isotropic and linear elastic characteristics of steel. One model established for evaluation of the web crippling limit state for steel beams assumes that elastic buckling is initiated in the web resulting in out of plane bending deformations resulting in a loss of the load carrying capacity of the web in a localized region. This behavior forces localized bending stress concentrations within the effected portion of the loaded flange and a plastic hinge mechanism is formed simultaneously in the flange and the webs [7]. Although elastic buckling failure is likely to be involved in the web crippling limit state for the hybrid-composite girder, the FRP materials are not elastic, perfectly plastic. Because these materials remain linear up to failure, a plastic hinge cannot form. This makes it more difficult to quantify the magnitude of the load that will initiate this type of failure.

Another factor influencing the web crippling limit state is the bond to the foam core. This bonding results in a significant increase in the elastic buckling strength of the webs with respect to vertical loads as well as shear buckling. The bonding also reduces the out of plane rotations that result in transverse bending stresses. Although the foam has much lower strength and stiffness than the FRP webs, the compressive strength of the core material is sufficient to withstand the vertical loads typical of a ballasted-deck bridge and transfer them to the compression strut, even without the FRP webs.

A rigorous analysis of the web-crippling behavior of the hybrid-composite beam is a research project in and of itself. Preliminary analysis of this limit state has been limited. The vertical stresses on the top of the beam have been calculated based on the allowable distribution length of axle loads as indicated in AREMA. In preliminary experimental tests, web crippling due to vertical stresses on the beam has not initiated a premature failure of the beam. Further investigation of this limit state is warranted to better understand the behavior of the webs.

4.3.7 Anchorage Zone Limit State

The anchorage zone limit state will depend on what type of tension reinforcing is used in the beam. Where carbon fibers are used for the reinforcing, it is preferable to place the carbon fibers directly adjacent to the glass fibers comprising the bottom flange. Running the carbon vertically up along the end of the beam would develop or anchor the tension force in the carbon. The compression reinforcing would simply terminate at the bottom corner of the beam. Analysis of the bending stresses in the carbon fibers around the corner of the beam could be a complex analysis problem. In depth analysis of the anchorage zone for carbon fiber tension reinforcing has not been evaluated to date.

For steel tension reinforcing, the anchorage is contingent on the type of steel used. For the most part it is preferable to use an anchorage device that is an adaptation of conventional anchorages used in post-tensioning systems. For the prototype beam, the type of steel selected was 150 ksi, high-strength post-tensioning bars. The conventional system for anchoring these bars, is to use a rectangular steel anchor plate bearing directly on concrete. The holes in the anchor plates are countersunk on one side to accommodate a spherical nut that is screwed onto threads that are deformed on the surfaces of the bars. The spherical nut allows for minor angle breaks at the anchorage location without introducing bending stress concentrations into the bars. In these type systems, the dimensions of the anchorage plate are determined in order to control the bearing stresses in the concrete. The thickness of the bearing plates is conservatively oversized to ensure that there is no yielding in the steel plates.

The anchorage designed for the prototype test specimen is comprised of two rectangular plates welded together in an "L" shape using fillet welds. The eight-inch long plate that sits on the bottom of the beam has edges that are machined to the same one-inch radius as the corners of the box beam. This prevents any discontinuities in the glass fabric at this location that could result in stress concentrations. The bottom plate also serves two other important functions. For one, it provides a sound bearing plate against the neoprene bearing supporting the beam. For another, it helps maintain the alignment of the other bearing plate that is anchoring the bars during the fabrication process,.

An important factor in the design of the anchorage system, regardless of the type of tension reinforcing used, is to make sure that all of the forces are equilibrated at a single work point. This is evident in the partial section of the beam shown previously in Figure 4. Note that the forces in the compression reinforcing, the tension reinforcing and the vertical bearing force are all coincident to work point 1(WP1). This is intentional to avoid any eccentricities in the resolution of these forces that could result in a rotation of the anchorage and a premature failure of the beam. It should also be evident that based on this simple free body diagram, showing the resolution of forces at WP1, the force on the back anchor plate is equal to the tension force in the bars. The back anchor plates have subsequently been designed to provide for a compressive stress that is less than the stress in the compression reinforcing to prevent a premature failure at this location.

4.3.8 Load & Resistance Factors

In developing a consistent design philosophy for the hybrid-composite beam system as it relates to allowable or ultimate strength, a logical argument can be made regarding a load and resistance factor (LRFD) design for the bending limit state. Meanwhile, serviceability is evaluated using unfactored loads, which is standard practice for most structures. Some of the other limit states such as shear and web crippling may lend themselves better to allowable stress design (ASD). It should be noted that when evaluating the performance of FRP materials, it is not unusual to use limiting strains rather than limiting stresses, as is typically done for steel and concrete structures. At this stage of development, it is recommended that ASD be adapted as a rational means of evaluating these structural members. At the same time, it is recommended that a USD, i.e. LRFD check be made for the bending limit state.

For the long-term development of the beam system, it may be preferable to develop the design equations strictly using an LRFD design philosophy, while still checking serviceability with unfactored loads. As noted in the report, where load and resistance factors have been used, they have been taken directly from Chapter 8 of the AREMA code. To formulate rational load and resistance factors a reliability analysis would need to be conducted for each limit state with respect to the statistical probability of the demand and capacity of the beam based on the materials used. Again, this would require a level of effort that is beyond the scope of this project. For now, it is anticipated that the load factors used from AREMA will result in a conservative design.

4.4 EXPERIMENTAL TEST PROGRAM

4.4.1 VARTM Manufacturing Process for Prototype Beam

Not only do FRP composites offer tremendous variations in materials and composition, but there are numerous manufacturing processes available to fabricate structural members out of these materials including: hand lay-up, pultrusions, filament winding, autoclaves and vacuum assisted resin transfer methods (VARTM) to name a few. Once again, a detailed discussion of the various processes that are available for composites is beyond the scope of this report. Instead, we will focus on the VARTM process that was selected for fabrication of the prototype test specimen.

The VARTM process has many advantages that include low tooling costs and the use of vinyl-ester resins that offer room temperature infusion and curing conditions. As a result, this process allows considerable flexibility in the size and shape of parts that can be manufactured. One proprietary VARTM process is known as the Seeman's Composite Resin Infusion Molding Process or SCRIMP. The specifics of this process will be described through the steps involved in fabricating the test specimen. The overall length of the beam was 19 feet (18 feet center to center of bearings). The beam was 24 inches tall and 12 inches wide. It was designed for Cooper E-80 (or alternate) live load, assuming that twelve such beams would support a ballasted deck. The steps for fabrication were as follows:

- A three-sided wooden box was fabricated to the dimensions of the beam as a low cost mold. The mold was positioned so that the beam would lay on its side with the flanges oriented vertically.

- Along the centerlines of the vertical walls for the flanges, slots were made along most of the length to install omega channels. These channels act as a line source for the resin infusion and vacuum.
- A thin plastic release film was bonded on the inner surfaces of the wooden mold.
- Five equally spaced nylon tee-connectors were placed in the omega tubes on each face to provide access for resin or vacuum source. The omega channels are then stapled inside the slots.
- A distribution media was placed and stapled over the entire mold surface followed by a layer of peel ply.
- One layer of QM6408 glass fabric was laid on the mold such that it mapped over three sides of the mold and overhung for wrapping the fourth side of the beam. The warp direction of the fabric was set along the longitudinal axis.
- A 22" wide strip of QM6408 was placed in the bottom of the mold so both webs would have two layers of fabric.
- The three, 1" diameter, 150 ksi, post-tensioning bars were positioned along the face to comprise the tension flange of the beam. The threaded bars were secured to the prefabricated steel anchorage with standard bar nuts. Five weldable strain gages (type CEA-06-W250-350) were attached to each bar at the centerline, quarter points and anchorages prior to fabrication.
- Blocks of polyisocyanurate foam were cut to fill the voided space in the mold while still leaving a cavity to accommodate the compression reinforcement. Spacers and clamps were used to ensure the dimensions of the cavity.
- The arch cavity was filled with Masterbuilders, MasterFlow 928, high-strength portland cement grout. Five concrete strain gages (type EGP-5-350) were embedded along the centerline of the compression reinforcing to line up with the steel gages.
- The overhanging QM6408 was wrapped over the open side of the mold as well as the peel ply and distribution media. The mold was closed with a pre-fabricated wooden cull plate and two bulkheads at the beam ends.
- The entire mold was then completely enclosed in two layers of vacuum bagging film sealed with tacky tape.
- Vacuum was applied through the five equally spaced vacuum ports. Vacuum debulking was applied and vacuum was held overnight to ensure good compaction.
- After evacuating all of the air the vinyl ester resin was pulled into the preform by the vacuum pressure. The resin used was a Dow Derakane 411-C50 mixed with 0.2% Co-Nap, 2% Trignoz and 0.05% 2, 4 Pentandion (inhibitor). 50 kg of resin was prepared to facilitate complete wet out and circulation for the anticipated 20 kg required.
- The beam was completely infused in about 45 minutes. The flow front was parallel to the infusion line. Minor race-tracking was observed at the beam ends, however with time the tiny dry spots became saturated. Resin coming out of the vacuum ports was re-circulated until the resin viscosity was high enough. The inlet was then closed and vacuum held overnight. The next morning the beam was ready to de-mold.
- Visual inspection of the beam revealed good wet out of the preform without any dry spots. Negligible traces of small undulations of the fabric on the beam surface were observed. The fabric in the contact region of compression reinforcing showed white color, which indicates a deficiency in the tow saturation. This can be prevented in the future by preventing infiltration of moisture from the compression reinforcing into the glass fabric prior to infusion.

Once the fabrication of the beam was complete, the beam was positioned in the load frame attached to the reaction floor in the Civil Engineering Lab at the University of Delaware. Appendix A contains a series of photographs that depict the experimental portion of this investigation from fabrication through load testing, including forensic photos.

4.4.2 Load Frame and Instrumentation for Prototype Beam

Load testing of the beam was facilitated by positioning the beam under a load frame supporting two 150 kip MTS, hydraulic actuators. The loading frame was anchored to a reaction floor. The beam was simply supported on 70 durometer neoprene pads positioned on two concrete blocks. The load was distributed out along the top of the beam using two levels of spreader beams separated by steel rollers. The final contact on the top of the beam consisted of one-foot square steel plates bearing on a one-inch strip of neoprene resting on the top flange of the beam. Each actuator load was thus distributed over a length of four feet, which approximates the longitudinal distribution length specified for a ballasted deck in the AREMA code.

The loading of the beam was performed using a Flex Test IIM, MTS hydraulic pump system. All tests were performed using force control loading conditions rather than displacement control. In addition to the strain gages installed on the tension and compression reinforcing described previously, longitudinal gages and rosette gages were attached to the glass shell. The rosettes were comprised of three-element, 60-degree delta single plane rosettes (CEA-06-250UY-350) bonded to the shell. Six Linear Variable Differential Transformers (LVDTs) were placed under the beam to monitor deflections and 3 LVDTs were placed at the east-end of the beam to monitor rotations. In all, 45 channels of data were monitored under each test using a Vishay 5000 data acquisition system connected to a personal computer. Figure 5 depicts an elevation of the test set-up for the beam for the symmetrical actuator position.

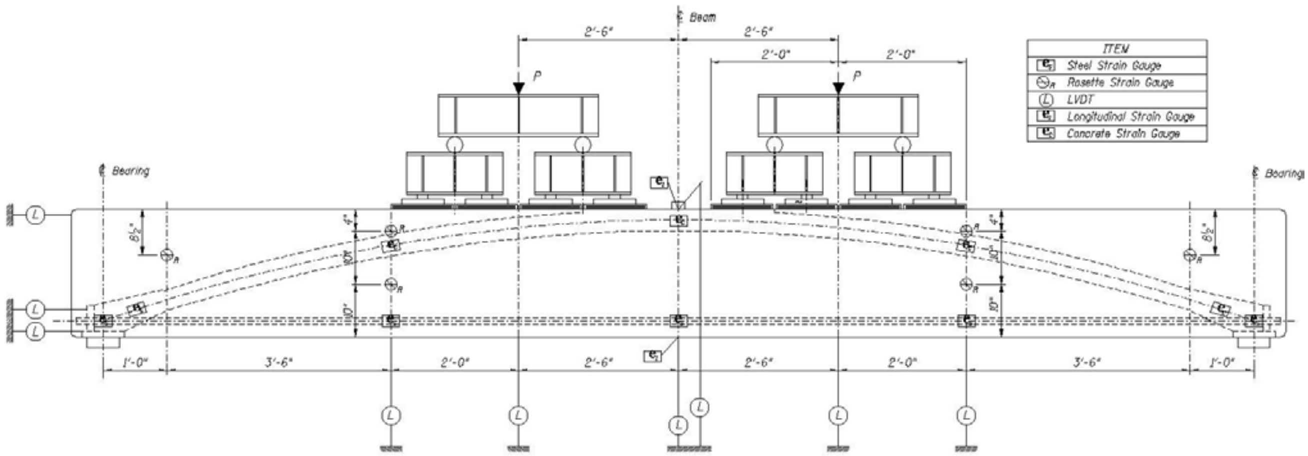


Figure 5. Elevation of Test Set-Up and Instrumentation Plan

4.4.3 Prototype Beam Load Test Patterns

A number of load tests were performed on the prototype beam to simulate the effects of Cooper E-80 locomotives both under symmetrical and unsymmetrical conditions. The following is a test matrix that indicates magnitudes of the actuator loads, in kilopounds (kips), and the duration of time, in minutes, between points of increasing, decreasing or constant load:

Test	Force	Dur.	Force	Dur.	Force	Dur.	Force	Dur.	Force	Dur.	Force	Dur.	Force
1S	0	5	1.84	10	1.84	5	16.76	5	16.76	5	0	-	-
2S	0	5	21.46	0	0	-	-	-	-	-	-	-	-
3S	0	5	1.84	5	1.84	5	16.76	5	16.76	5	33.2	5	0
4U	0	5	1.84	5	1.84	5	16.76	5	16.76	1	0	-	-
5U	0	5	1.84	5	1.84	5	16.76	5	16.76	5	0	-	-
6C	100,000 cycles @ 2 Hz, symmetrical loading varying between 11.63 kips and 16.76 kips per actuator (2 each)												
7S	0	5	1.84	5	1.84	5	16.76	5	16.76	5	33.2	5	0
8S	0	14	45.81	-	-	-	-	-	-	-	-	-	-
9S	45.86	5	58.74	-	-	-	-	-	-	-	-	-	-

Table 7. Load Test Matrix

In general two different load configurations were evaluated. In the symmetrical load tests, denoted with an "S" suffix, the two actuators were placed symmetrically about the centerline of the beam. In the unsymmetrical load tests, denoted with a "U" suffix, the two actuators were positioned on one half of the beam to simulate the maximum service level shear force anticipated in the beam. In both cases the 1.84 kip/actuator load is intended to simulate the superimposed dead load resulting from a ballasted deck and track work. Likewise, the 16.76 kip/actuator loads are devised to produce the same maximum moment in the beam as Cooper E-80 axle loads with impact. The 33.2 kip/actuator load evident in tests 3S and 7S are intended to simulate the maximum moment in the span resulting from the factored loads based on LFD Group I load factors for concrete bridges as specified in Chapter 8 of the AREMA code.

The cyclical load case, denoted with the "C" suffix was devised to simulate the stress range induced in a beam subjected to repeated Cooper E-80 axle loads traveling across the bridge. No data was collected for the cyclical load test.

The main purpose was to confirm that the elastic behavior would not change after numerous cycles of loading. In that regard the test was successful. The last two tests were conducted to determine the actuator loads at which the girder would no longer be able to sustain load, i.e. ultimate capacity.

4.4.4 Correlation of Test Data to Predicted Results

Prior to initiating the load tests, theoretical predictions of the beam behavior were calculated relative to the various load configurations and load magnitudes. The behavior of the beam under loading was very close to the predicted results. In general, the beam exhibited linear elastic behavior almost up to the ultimate load as anticipated for a serviceability controlled design. The calculated failure load for the beam was based on reaching a compression limit strain of 0.003 in the high strength grout compression reinforcement. Other premature failure modes that were anticipated included crushing failure of the core material and de-bonding of the glass webs from the core material or compression reinforcing.

Before discussing the data compiled from the tests, it is worth noting a few observations made during the loading cycles. During the first loading cycle where an increase in the maximum load was initiated, it was not unusual to here some acoustic emissions from the beam. These popping noises were relatively loud but were not evident from a review of the test data. It is not unusual to hear this sort of popping on FRP structural elements during the first load cycle. The noises generally result from internal settlement of the components and are amplified by the box itself. In almost all cases, when the beam was reloaded on subsequent tests, no acoustic emissions were heard at load levels below the previous maximum load.

Since the specified compressive strength of the foam core is 50 psi, it was anticipated that once the actuator loads approached 30 kips, some crushing of the core material might be evident. In fact it was evident that at the higher loads, the core material did begin to yield. This crushing resulted in some localized transverse bending stresses in the glass webs as well as a redistribution of the bearing stresses that caused a stress concentration at the contact point approximately 3 feet from the centerline of the bridge. This is approximately the tangency point of the compression reinforcing. Crushing of the core material could have contributed to the failure mode prior to reaching the predicted compression strain in the concrete. It should be noted that this localized failure can be avoided with a slight modification to the detailing at the top flange interface of the foam and the compression reinforcing.

The primary indicator of predictable behavior during the load test was deflection. Again, the main load carrying components of the beam were designed to meet the live load deflection criteria of span/640 according to the AREMA code. The predicted deflection under the design service load was 0.35 inches. The measured deflection was 0.43 inches. The slightly higher deflection could be attributed to slightly lower elastic modulus in the grout, variations in the depth of the compression rib or a slight reduction in the distance between the compression and tension reinforcing. Although the measured deflection was 23% greater than predicted, the difference is only 1/16 inch. The total deflection at the factored demand was 0.88 inches where the predicted value was 0.78 inches. Again the same considerations may have contributed to the slightly higher deflection.

The behavioral characteristic that posed the greatest uncertainty during the analysis stage was shear. The tests indicate that the girder actually behaves like a beam up to and well beyond the factored demand. That is to say that the axial force components in the compression and tension reinforcing are transferred through shear in the webs. This behavior is confirmed by the fact that the strain in the compression reinforcing and tension reinforcing drops to nearly zero at the anchorages. Had the strain remained constant along the length of the compression and tension reinforcing, the behavior would have been more akin to a tied arch. Either behavior provides an acceptable internal load path.

One of the unique benefits of the hybrid-composite girder is the ability to function as a tied arch. As a result, the girder has a built in redundant load path. The final test demonstrates this behavior in that as the actuator loads exceed 40 kips, the load path starts to shift from shear in the webs to a more uniform distribution of force in the steel and grout, similar to arch behavior. The other assumption that was confirmed is that the bonding of the glass webs to the core provided the necessary restraint to prevent elastic shear buckling of the webs at a much lower stress level. This ensures a more efficient use of the materials for shear. A graphical representation of the strains and deflections at various points in the beam can be seen in the test data depicted in Figures 6 and 7 for load tests 7S and 9S respectively.

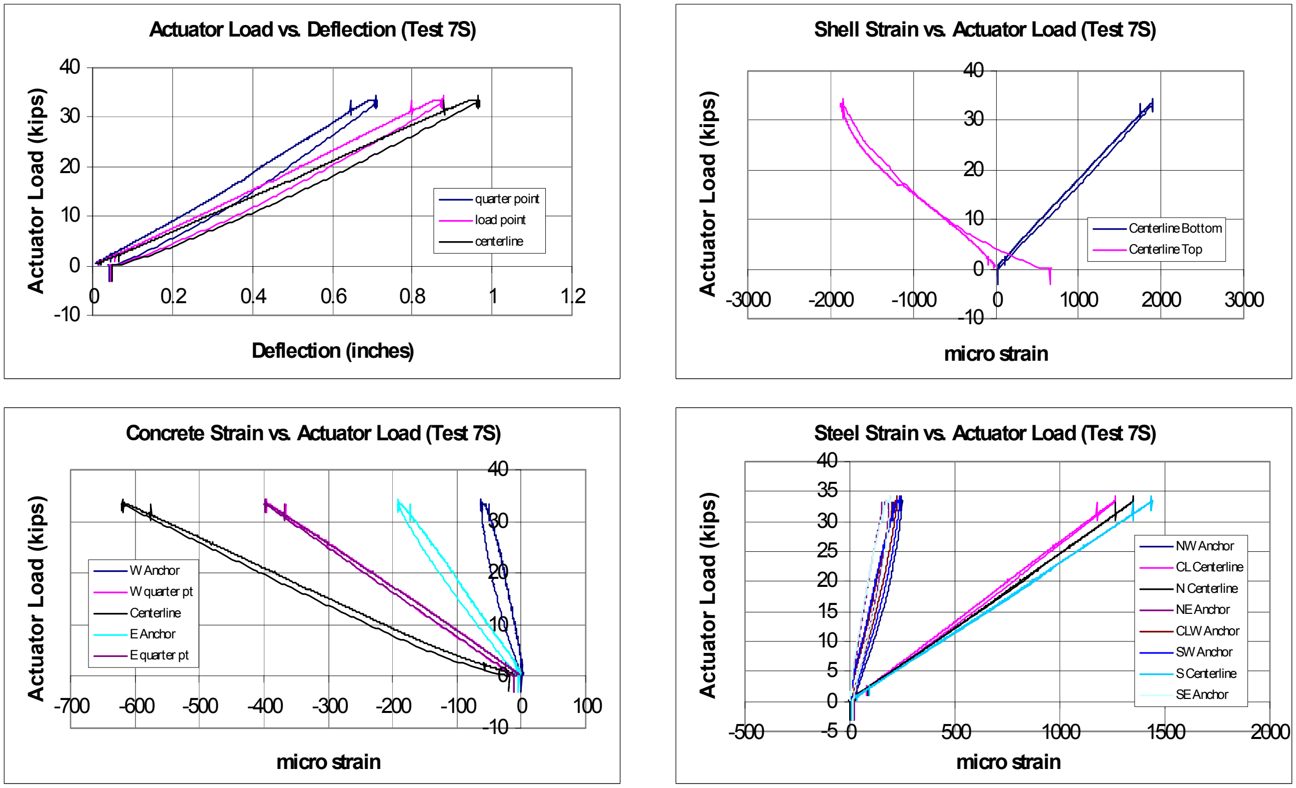


Figure 6. Test Data from Symmetrical Load Test 7S (Factored Demand)

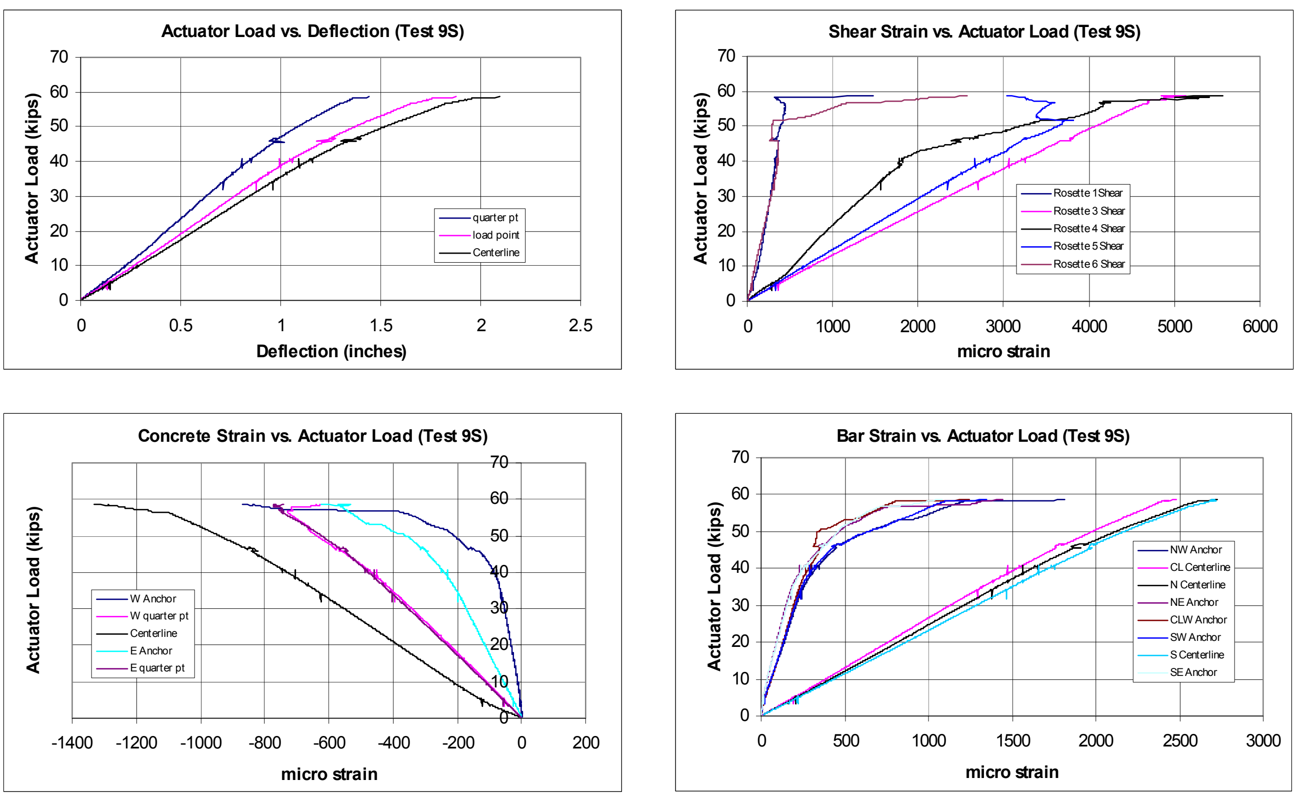


Figure 7. Test Data from Symmetrical Load Test 9S (Ultimate)

5.0 PLANS FOR IMPLEMENTATION

5.1 TYPE 2 PROTOTYPE DEMONSTRATION

The objective of this Type 1 exploration was to investigate the design limit states, evaluate the cost metrics, fabricate and test a prototype beam to verify the structural behavior and manufacturing techniques for the IDEA product. As this investigation comes to a close, all of these objectives have been successfully achieved. The next logical step is to demonstrate the viability of the product by constructing a serviceable bridge using the hybrid-composite beams. The focus of a Type 2 investigation would be a product application comprised of the fabrication and installation of an entire span on a railroad bridge to confirm and monitor the in service behavior.

5.2 STANDARDIZATION OF ERECTION DETAILS

Although the hybrid-composite girder is a fairly radical departure from conventional bridge structures, the intended erection methods to be used are not much different than those used for concrete and steel beams. This is no coincidence. The hybrid-composite structural framing system has been developed so that once delivered to the construction site, there is no special equipment or knowledge of advanced composite materials required of the contractor. The proposed framing system utilizing the hybrid-composite girder system should work equally well on horizontally and vertically curved alignments as well as skewed spans.

One unique characteristic of the hybrid-girder is that it is possible to inject the compression reinforcing once the girders have been erected. The advantage of introducing the compression reinforcing insitu is that the self weight of the beam for shipping and erection purposes is approximately 1/5 the weight of a comparable steel framed system and approximately 1/12 the weight of a comparable precast box beam system. This dramatic reduction in weight can result in a reduction in the size and cost of the equipment necessary for erection. This has been considered in evaluating the erection costs for the hybrid composite girders.

In evaluating the framing systems for railroad bridges, it is evident that the more efficient use of the hybrid-composite girders results from placing the girders side by side. This is similar to what is done for prestressed concrete box girder bridges as well. Arranging the girders in this manner not only provides for the shallowest depth on the overall structure, but also eliminates the need for any intermediate diaphragms that might be required for bracing adjacent girders. It also provides for a closed surface for placement of the ballast and track work.

The girders are the primary load-carrying element of the bridge superstructure. However, there are other components of the superstructure that have to be compatible with the framing system used. Other components that need to be considered include support bearings, expansion joints, the deck surface and parapets necessary to retain the ballast and track work. In some instances is will also be necessary to provide diaphragms or lateral bracing at the ends of the spans or at intermediate points. In some circumstances it may also be desirable to provide seismic restrainers at the ends of the beams. Further development of these types of details should focus on providing details that are similar and where possible, compatible with conventional bridge technology.

5.3 PRODUCT COMMERCIALIZATION

The ability to manufacture a hybrid-composite beam to satisfy AREMA design criteria for a Cooper E-80 locomotive was demonstrated through this Type 1 exploration. The success of this investigation should be further evident by the fact that only one beam had to be manufactured and tested to satisfy the design and manufacturing criteria. The main objective of the Type 2 Product Application will be to demonstrate the viability of the hybrid-composite girder as an alternate framing system to steel and concrete beams. This objective will be met by the construction of an entire span on a railroad bridge to document a historical record of both structural and cost performance. The success of a Type 2 investigation should provide both the end users and potential manufacturers with the increased confidence to embrace this new technology and help facilitate the acceptance of the hybrid-composite beam as a means to improve quality and longevity in reconstruction of our nation's infrastructure.

6.0 CONCLUSIONS

Various conclusions can be drawn from each stage of this investigation. With respect to the cost metrics, it appears that the economic advantages of the hybrid-composite girders are a function of span length. For shorter spans, the hybrid-composite girders appear to be less costly than steel, whereas for longer spans, the lightweight advantages of the hybrid-girder may result in the girder costs being less than for prestressed concrete beams. In any event, it appears that a span length of approximately 30 feet represents a significant portion of the railroad bridge inventory and replacement market, and a logical target span length to focus on for further development.

With respect to design, the primary conclusion regarding the structural behavior of the hybrid-composite beam system is that serviceability appears to consistently govern. A spreadsheet was developed to facilitate the simultaneous evaluation of the design limit states for the critical components. Throughout the development, it was possible to begin formulating some intuitive knowledge that can be used in expediting the design for a specific application. The following is a brief listing of the steps that can guide the designer in making decisions regarding the composition of the beam to arrive at a satisfactory design.

- Establish geometrics of structural cross section.
- Make initial assumption regarding depth of girders (span/10 for railroad bridges).
- Make initial assumption of girder width based on depth (generally depth/3 to depth/2). The out to out dimensions of girder help establish number of girders for structural cross section
- Make preliminary assumption on thickness for FRP web and flange components. It is helpful if the designer has previous knowledge of the approximate thickness of fabricated FRP laminates per layer of fabric.
- Adjust dimensions of compression conduit and amount of tension reinforcing until deflection criteria are met.
- Check ultimate moment capacity to make sure strength is satisfied. Adjust compression and tension reinforcing as required.
- Check shear capacity at 1/10 points along the beam. Increase web thickness if necessary.
- Calculate vertical stress in beam and compare to capacity of core material and webs.
- Calculate stresses at anchorage.
- Establish geometry at anchorage zone to equilibrate bearing reaction with compression and tension reinforcing without introducing undesirable eccentricities

Other conclusions that can be drawn from the analytical and experimental work are related to the evaluation of load sharing of the shear forces between the compression reinforcing and the FRP webs. Again it was evident from the testing, that although the shear in the beams is carried by the FRP webs, the arching action of the compression reinforcing offers a redundant load path that is exploited as the applied forces exceed the factored demand.

Despite the success of this investigation there are still areas that warrant further investigation and refinement. One aspect that warrants further investigation is the relative economic differences between carbon fiber and steel tension reinforcing. Based on the structural performance criteria and the fact that the tension reinforcing only accounts for roughly 25% of the total material cost of the beam, it appears carbon fibers may still prove to be economically viable. Another area that warrants further investigation is fine tuning the tooling and lay-up using the recyclable mandrel material to facilitate lighter weight for shipping and erection. Refinements will also evolve in the analysis and design methodologies as is inevitable with any material.

Input has also been solicited from members of the engineering community affiliated with the Class 1 Railroads. Some of the issues and concerns raised by these individuals that warrant further investigation include:

- Vulnerability to damage and methodologies for repair and maintenance.
- Ultraviolet stability for 80 plus years of service is desirable.
- Additional load testing to better quantify shear strength, crushing/buckling of the webs and fatigue stress limit states (2 Million Cycles as a minimum must be considered in fatigue evaluation).
- Testing of multiple beam systems to ascertain distribution of loads to the plurality of beams in the bridge cross-section.
- Effects of thermal cycles.
- Further establishment and documentation of non-destructive evaluation techniques.

Many of these questions can be or have already been addressed with the evolution of advanced composite materials. The end users as well as the engineering community supporting these users should be consulted on developing future experimental testing programs to ensure that the critical concerns are addressed. In addition to the technical issues, several representatives of the member railroads of the AAR have expressed concerns regarding the proprietary nature of the hybrid-composite beams. Careful consideration will have to be given in developing the manufacturing and procurement model for this product in order to alleviate any concerns about impropriety regarding the procurement.

The most important conclusion is simply that the concept behind this IDEA product works. The experimental stage confirmed the ability to design and manufacture a hybrid-composite beam that can be used as a framing member in a railroad bridge. Additional testing should be performed to confirm that these results can be consistently reproduced. However the engineering principles defining the structural behavior are as simplistic as the tooling and manufacturing process used to fabricate the beams.

7.0 INVESTIGATOR PROFILE

John R. Hillman (P.E., S.E.) In addition to being the sole inventor of *the "Plasticon-Optimized Composite Beam System," United States Patent Number 6,145,270*, Mr. Hillman is a Senior Associate with Teng & Associates, Inc., Chicago, IL. Mr. Hillman received his Masters of Science in Civil Engineering from Virginia Polytechnic Institute and State University (VPI), Blacksburg, VA. While at VPI, his research involved developing Innovative Light-Weight Floor Systems for Steel Framed Buildings as part of a research project funded by the American Institute of Steel Construction (AISC). It was during the course of this project that Mr. Hillman got his first exposure to FRP structures. Over the course of the past fifteen years, Mr. Hillman has been employed as a structural engineer in the inspection, construction and design of unique bridges. His background includes construction engineering and inspection of a cable stayed bridge, project management for the construction of an incrementally launched concrete box-girder bridge, as well as the design of; segmental box girder bridges, cable stayed bridges, composite steel/post-tensioned concrete structures and steel arch bridges. Prior to joining Teng & Associates, he was employed by J. Muller International, where he worked in close coordination with Hardcore DuPont as Project Manager for the design of the 75-foot Magazine Ditch (FRP) Bridge for the Delaware River and Bay Authority. Mr. Hillman is a registered Professional Engineer in the States of Florida and Indiana as well as with NCEES. He is also a registered Structural Engineer in the State of Illinois.

REFERENCES

[1] *The Status of the Nation's Highway Bridges: Highway Bridge Rehabilitation Program and National Bridge Inventory,Thirteenth Report to the United States Concress,* U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., May 1997.

[2] Ehlen, Mark A. and Marshall, Harold E., *The Economics of New-Technology Materials: A Case Study of FRP Bridge Decking, NISTIR 5864,* U.S. Department of Commerce Technology Administration, National Institute of Standards and Technology, July 1996.

[3] MacGregor, James G., "*Reinforced Concrete – Mechanics and Design",* Prentice Hall, 1988.

[4] Demitz, John R., "*CCM Report 99-03, Limit States Design Methodology for Composite Material Bridge Structures",* Masters Thesis, University of Delaware, 1999.

[5] Timeshenko, S. P., and Gere, J. M. "Theory of Elastic Stability" McGraw-Hill Book Co., New York, NY, 1961.

[6] *"FHWA - TS-82-216, Stuctural Plastics Design Manual – Phases 2 and 3; Chapters 5-10",* Structural Plastics Research Council, American Society of Civil Engineers, September 1981.

[7] Roberts, T.M. and Newark, A.C.B., *Strength of Webs Subjected to Compressive Edge Loading,* American Society of Civil Engineers, Journal of Structural Engineering, pp 176-183, February 1997.